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PERFORMANCE OF BOLTED WOOD-TO-CONCRETE CONNECTIONS AND BOLTED CONNECTIONS IN PLYWOOD SHEAR WALLS

A Major Report

by

DONALD ANTHONY JONES

Submitted to the Office of Graduate Studies of
Texas A&M University
in partial fulfillment of the requirements for the degree of

MASTER OF ENGINEERING

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ABSTRACT

The Performance of Bolted Wood-to-Concrete Connections and Bolted Connections in Plywood Shear Walls. (August 1996)

Donald Anthony Jones, B.S., United States Air Force Academy

Chair of Advisory Committee: Dr. Joseph M. Bracci

In the aftermath of recent earthquakes in populated areas, observations of wood-frame residential and commercial construction have revealed damage to plywood shear walls in the form of splitting of the sill plate along the line of anchor bolts in the foundation and of the end post along the bolts in the hold-down connection. This paper presents an experimental investigation of the sill plate connection, using a variety of bolt sizes and member thicknesses. The investigation also includes full-scale studies of plywood shear wall assemblies. Standard connections are examined as well as connections strengthened with reinforcing clamps. These clamps are shown to increase strength and energy absorption in wood-to-concrete and wood-to-wood connections where the failure mode is splitting of the wood member. The reinforcing clamps are shown to increase the stiffness and energy dissipation of a shear wall specimen.

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CHAPTER I

INTRODUCTION

1.1 Background

Timber structures have been observed to exhibit good performance in small to moderate earthquakes. The good performance is due to the redundancy of the structures, the high strength-to-weight ratio of wood, and the inherent ductility of the fasteners used in construction. Observations from recent earthquakes, however, have led to concerns regarding the performance of wood frame structures. Discontinuous shear walls and irregular shapes have reduced the lateral force resistance of the structure. Larger buildings with concrete floors and roofs, along with heavier materials used for fire control and aesthetics, combine to increase the inertial forces experienced in an earthquake.

One estimate places the current value of wood-framed residential construction in California at over 100 billion dollars (Foliente 1994). The 1994 Northridge earthquake produced insured losses of almost 10 billion dollars. Since only about 40% of homeowners carry earthquake insurance, the actual losses surely exceed $20 billion. Because of the devastating potential of an earthquake in a heavily populated region, many companies refuse to provide homeowner's insurance. To help reduce the damages and protect both homeowners and insurance companies, constant improvement should be made in wood frame construction.

The lateral seismic forces in a timber structure are resisted primarily by horizontal diaphragms, such as floors, and vertical diaphragms, such as shear walls. The focus of this study is the deformation and load response behavior of shear walls. The intended

performance of these components is based on a ductile failure, with gradual slippage of the nails around the perimeter of the sheathing. However, excessive deformations can still lead to high expense as interior components are damaged by the movement. The entire shear wall subassembly must also effectively transfer forces through the base plate to the foundation, through embedded anchor bolts. Excessive slip in these connections can lead to unacceptable movement of the entire structure.

Deformation of shear walls consists of four main components (See Fig. 1.1): (i) shear deformation (known as racking deformation), when the nails at the exterior of the plywood panel slip, causing the whole panel to rotate as a unit; (ii) sliding deformation, when the entire wall slides laterally because of slippage at the connection to the foundation; (iii) overturning deformation, when one side of the wall raises up, making the entire assembly rotate; and (iv) bending deformation, which is usually a small component. The bending component is prominent in tall, narrow shear walls. Longer walls will be more susceptible to sliding deformation, since the lateral force is high in comparison to the overturning moment. Of particular interest to this study is the performance of the bolted connections, and their associated deformations. The sill plate is bolted to the foundation with anchor bolts, and hold-down connectors bolted to the wall ends are used in order to transfer uplift forces to the end posts. Therefore, the overturning displacement and the sliding displacement are the deformation components most influenced by bolted connections.

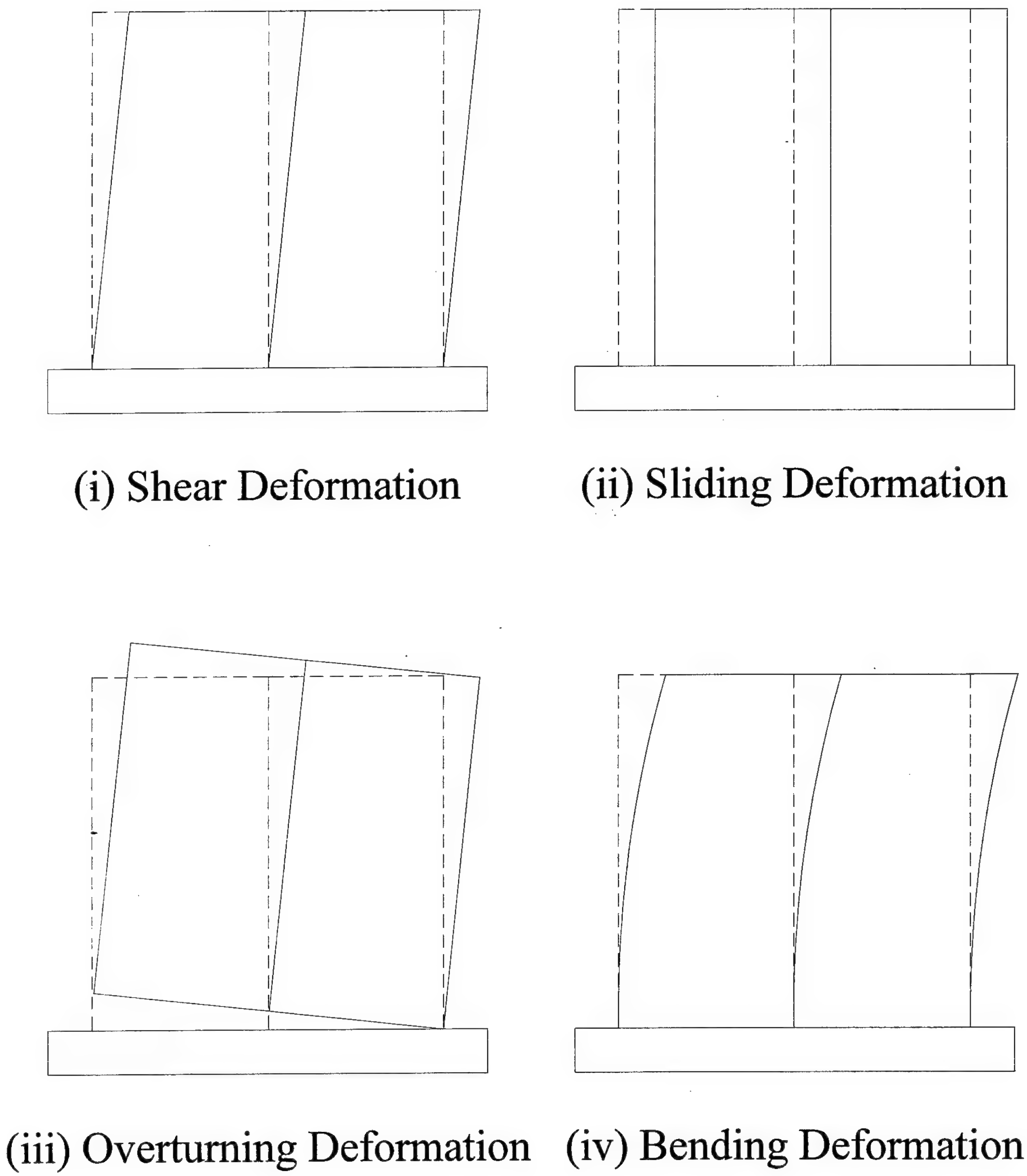


FIG. 1.1. Components of Deflection in a Wood-Frame Shear Wall

Hold-down slip connectors can have a direct influence on wall deflections, as the uplift of the end post leads to deformation at the top of the wall. This deformation also increases the force on the bottom nails of the sheathing, reducing their capacity for resisting racking deformation (Rose 1995). If overturning displacements are large compared to the shear displacements, the wall is not allowed to develop it's full lateral load capacity. Therefore, reducing uplift in the tension studs is vital to maintaining the calculated strength and stiffness of the wall.

Recent earthquakes in populated areas have highlighted problems with the connection of the sill plate to the foundation. Filiatrault (1995) noted many single story houses were moved off of their foundation when sill plate anchorages failed. There was splitting of the sill plate or inadequate anchor bolt spacing to handle shear forces. When a house is moved off of its foundation, electrical, sewer, water, and most importantly gas connections can be damaged. The cost to move these structures back onto their foundations and repair the resultant damage is considerable. Hamburger (1994) noted that splitting of the sill plate was aggravated by close bolt spacing. When bolt holes are drilled close to one another in a wood member the allowable strength of the connection is lowered. Improving these individual connections will allow for safer structures, and preventing splitting will greatly improve the strength and ductility of the system.

The yield behavior of bolted wood connections is described by the National Design Specification for Wood Construction (NDS) (American Forest & Paper Association 1991). The NDS prescribes four main yield modes for bolted connections (See Figure 1.2). Yield modes I and II are dominated by crushing of the wood fibers

which are bearing against the steel fastener. Yield modes III and IV are characterized by local crushing of the wood members accompanied by the formation of plastic hinges in the fastener. The most brittle failures are usually from yield modes I and II, because the direct bearing of the bolt will tend to cause splitting of the wood member. The yielding of the fastener in modes III and IV usually produces a more ductile failure, low cycle fatigue of anchor bolts. Yield modes III and IV are more common and desirable in modern wood structures.

To improve the performance of these connections under extreme loading, such as the high lateral loading experienced in an earthquake, strength and ultimate deformation capacity should be maximized. Splitting of the wood member should be prevented, and the most ductile yield mode should be enforced. Splitting drastically reduces the load capacity of the connection, and no additional energy can be dissipated. The formation of plastic hinges in the bolt dissipates a great deal of energy, so it is more desirable than splitting of the wood member.

1.2 Literature Review

Most wood design philosophy is based on member design strengths derived from tests employing monotonic loading, as are the tests use to determine the resistance of shear panels. More research into the behavior of these structures under cyclic loading is being performed in order to more closely model the forces experienced in an earthquake. An example is the finite element model of a shear wall system proposed by White and

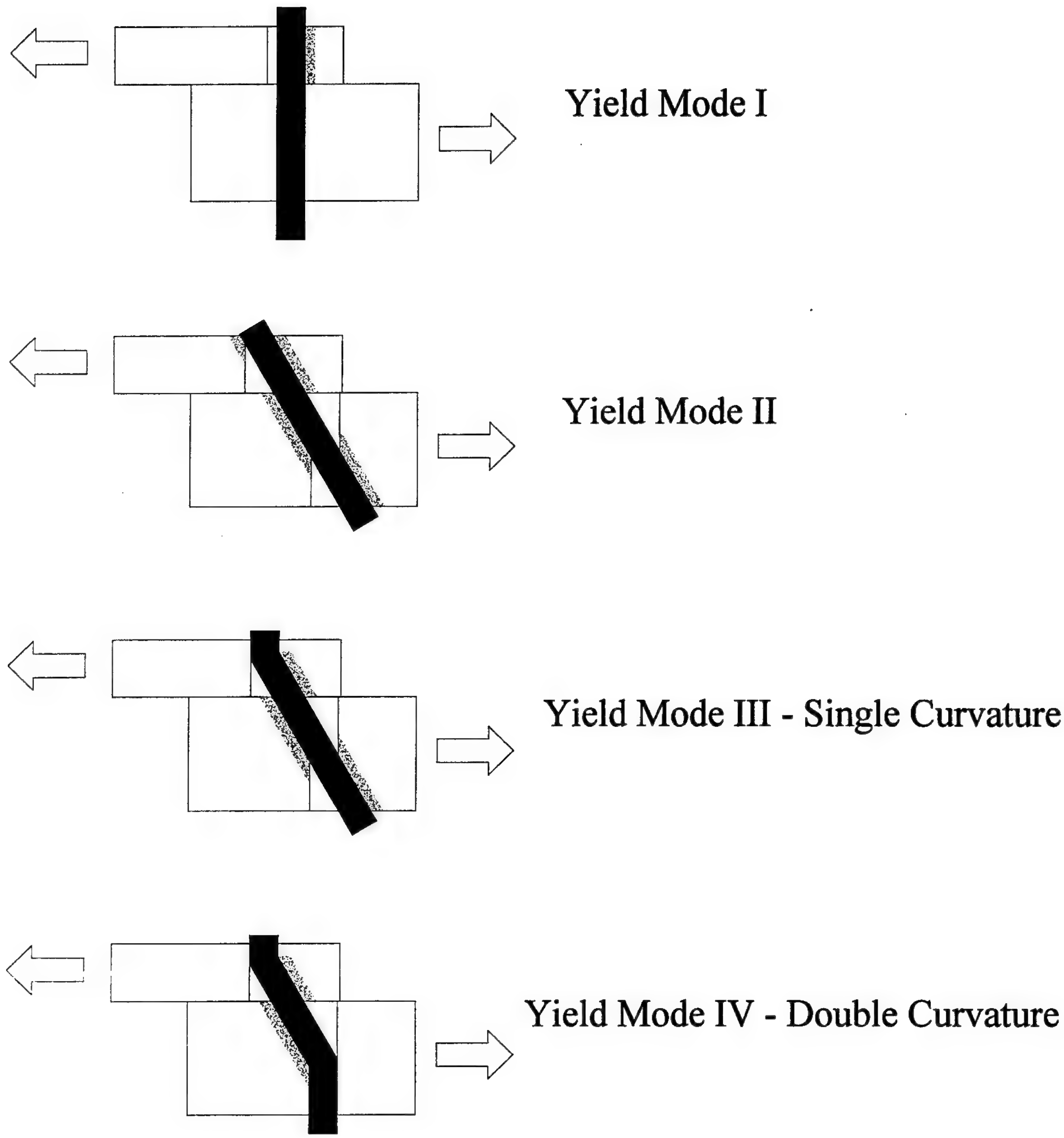


FIG. 1.2. Connection Yield Modes (NDS 1991)

Dolan (1995). This model can be used to examine the effects of shear wall openings and aspect ratio changes on a dynamically loaded shear wall. It does not, however, take into account the connection of the shear wall to the foundation. The recent surge in research indicates an increasing interest in the design and performance of wood structures, rather than relying on traditional construction practices.

The American Plywood Association (APA 1995) sponsored dynamic tests of plywood shear walls after the Northridge Earthquake. These tests, performed at the University of California, Irvine, are some of the first tests of shear wall assemblies under cyclic loading. These tests will help set the standard for cyclic loading of structural assemblies. Preliminary conclusions from the tests show that shear walls subjected to cyclic loads have about 20% less load capacity compared to shear walls in monotonic load tests. The performance was also thought to be contingent on overturning displacement. In order to minimize overturning deformation, special hold-downs were used on the end studs. In this report, a conservative "backbone curve" simulating the load-displacement relationship of a shear wall assembly is derived. The results of this test are contingent on the connectors and elements of the tested assembly, and further research is needed for other types of walls.

There are two ASTM standards which directly address wood frame shear walls. ASTM E72-80 describes the test methods which should be used to evaluate the racking load of a wood paneled assembly. This standard describes a monotonic load procedure and requires special hold-down rods which prevent uplift at the ends of the wall. Therefore, a major component of deflection is artificially constrained. ASTM E564-76

describes a static load procedure specifically for shear walls, including the positioning of instrumentation, but again does not address cyclic loading.

Research has also been done at Texas A&M University (Stromatt 1996) on the performance of 2x4 and 2x6 lumber in the sill plate connection. Investigations of these connections under quasi-static loading show that splitting of the wood member due to the prying action of the bolt may develop, which was the same action observed during past earthquakes. Retrofit sill plates, using reinforcing straps and clamps which provide confinement to the wood member, were shown to improve the deformation capability and energy dissipation capacity of the connection.

1.3 Research Objectives

The objective of this study was to evaluate the connections which are responsible for transferring shear forces along a plywood shear wall to the foundation. The connection of the base plate to a concrete foundation were closely examined with and without confining clamps. Full-scale shear wall models were also tested with and without confining straps at the bolted connections. The tests determined the contribution of the bolted connections on the sill plate and the end post to the overall wall performance at the yield and ultimate limit states. The tests also determined how wall performance is altered by the inclusion of the confining clamps. The performance measures on which this report focuses are strength, stiffness, deformation capability, and energy dissipation capability.

CHAPTER II

DESIGN AND PERFORMANCE OF WOOD FRAME SHEAR WALLS

2.1 Design Process

In order to design the lateral force resisting system of a timber structure, the first step is to analyze the forces acting on the structure. Load combinations are given by UBC (1994) Chapter 23 or ASCE 7-95. The procedures for evaluating the separate loads such as dead, live, earthquake or wind forces are given in the same publications. Of special consideration to this report are seismic forces, which constitute the controlling lateral forces for timber structures in seismic regions.

In order to analyze the seismic forces, there are two main methods, dynamic modal analysis and the static lateral force procedures. According to the requirements given in the UBC, the static lateral force procedure can be used for all structures which are under 65 feet in height. Most commercial and residential wood construction falls into this category, so the static lateral force procedure is usually used. A good description of this method is provided by Breyer (1993). Essentially, the lateral load is caused by the inertial effect of the roof or floors above. Therefore, the lateral load is directly related to the weight of the structure.

Lateral force resisting systems fall into three main categories: (i) rigid, moment resisting frames; (ii) braced frames; or (iii) shear walls. The majority of wood construction, especially residential, uses shear walls.

Shear walls use rigid structural panels to resist lateral forces and act as vertical cantilevers with the span equal to the height of the wall. The depth of these members parallel to the applied force is usually large in comparison to the height, so shear deformations rather than bending deformations govern the response. In typical construction, lateral forces are carried along horizontal diaphragms (floors) to the vertical diaphragms (walls), and then to the concrete foundation through various anchorage systems. The design of the shear wall to safely transmit these forces to the foundation is the focus of this research.

2.2 Shear Wall Design

The design of shear walls is covered by Breyer (1993) as well as by the APA's Design/Construction Guide -- Residential and Commercial (1994). Considerations in the design are the sheathing thickness, nailing schedule, chord design, strut design, aspect ratio (width-to-height ratio) of the shear wall, and anchorage requirements. The forces are transferred from the horizontal diaphragms to the wall by the struts, where the shear panel resists the shearing forces and the chords (end posts) resist the bending moment. The forces are transferred to the concrete by means of anchor bolts and hold-down connectors.

The shear panel is made of plywood or other structural panels, gypsum drywall, stucco (interior and exterior plaster), fiberboard or lumber sheathing. The resistances of plywood and other UBC rated panels, gypsum drywall, and fiberboard are tabulated in

various UBC tables. APA structural-rated sheathing tables have strengths tabulated in APA publications. Stucco resistances are provided by various manufacturers.

2.3 Shear Wall Performance

Filiatrault (1995) noted many damaged wood framed buildings after the 1994 Northridge earthquake in southern California. First stories and cripple walls were often much less stiff than the supported stories due to inadequate lateral bracing and open construction. These soft stories caused collapse when they were unable to resist the lateral forces imposed by the floors above.

After the recent Northridge earthquake, the Los Angeles Department of Building and Safety (1994) elected to reduce the allowable stresses listed in UBC tables for stucco and drywall. They have also elected to disallow the use of these materials as lateral force resisting elements at the ground floor of multi-level buildings. According to Hamburger (1994), stucco has shown to be extremely vulnerable to shear failure during the extreme loading imposed by an earthquake. Stucco walls have a wire mesh backed with building paper, which is stapled to the wall and then covered with plaster. Either the staples do not effectively fasten the stucco to the wall and shears away from the wall, or the mesh is securely fastened and the plaster cannot bond effectively, resulting in large scale spalling. Gypsum drywall degrades quickly under cyclic loading, so the material loses lateral force resisting capability. For this reason gypsum drywall is not considered to increase the shear resistance of wood frame walls designed for earthquake loads.

The City of Los Angeles (1994) has also elected to: (i) allow only 75% of the tabulated strength values for plywood shear walls; (ii) require the use of common nails, rather than box nails, on these assemblies; (iii) reduce the UBC minimum height to depth ratio from 3.5:1 to 2:1; and (iv) require 3x members between adjacent shear panels, or two 2x members joined together for earthquake repair work. The City of Los Angeles also noted deficiencies in the design of shear walls on hillside structures, where different heights of the wall produce shear concentrations. New regulations enacted after the 1994 Northridge earthquake require redundant lateral force resistant systems and require each section of a stepped wall to be anchored for both shear and uplift.

Hamburger (1994) noted failure of the nailing along the bottom of walls and to the boundary studs. This was caused by improper edge distance on the plywood as well as overdriving of the nails through the face ply of the sheathing. This failure can also be caused by excessive deformation in the hold-down, which puts additional uplift stresses on the bottom fasteners.

Filiatrault (1990) proposed a new concept for the earthquake resistant design of timber shear walls by providing friction devices at the corners of a framing system, which will theoretically increase the earthquake resistance and damage control potential of the system. The slipping of these friction devices should dissipate a large portion of the seismic energy instead of inelastic deformation of the sheathing-to-framing connectors. This system may be conveniently incorporated into existing structures to significantly upgrade their earthquake resistance.

2.4 Anchorage Design

The lateral force demands experienced by a shear wall in an earthquake are carried to the concrete foundation by means of anchor bolt connections. The procedure for calculating the number of anchor bolts is based on UBC tables for the dowel bearing strength of anchor bolts in the concrete and NDS methods for calculating the shear resistance of the bolt bearing on the wood sill plate. The UBC requires a minimum spacing of 6 feet on center (o/c) for anchor bolts. Uplift forces from the overturning moment demand on the wall are transferred from the tension chord to the foundation with a prefabricated metal bracket. These hold-down connectors are rated by the manufacturer for design load capacity.

2.5 Anchorage Performance

According to the Earthquake Engineering Research Institute's reconnaissance report of the 1994 Northridge Earthquake (1996), approximately 60,000 residential units experienced significant damage. Some causes of damage include longitudinal splitting in 2x4 and 2x6 sill plates of one story buildings. Hundreds of sill plates under shear walls failed in this fashion.

Hamburger (1994) performed an investigation of damaged wood structures after the Northridge earthquake and concluded similar findings. Slender shear walls performed poorly, although much better than the gypsum or stucco constructions. A significant amount of the deformation of the shear walls was concentrated at the base, which led to stretching of the hold down attachments and sliding of the sill plate. Because of this

sliding, there was splitting of the sill plate along the line of the anchor bolts. This splitting was aggravated by the close spacing of anchor bolts in the foundation and bolt holes which were drilled too large for the bolts. Bolts which did not line up with the drilled holes in the sill plate were often bent by the contractor to fit the holes. When bolts were tightened they straightened, damaging the sill plate. Bolts placed too close to the edge of the foundation pulled out of the side of the concrete. The end post often split at the attachment of the hold downs due to the overturning moment. This was due to the inadequacy of the stud, placement of the hold down too close to the edge of the member, and general overstressing of the member.

After the Northridge earthquake the city of Los Angeles (1994) specified that only 75% of the manufacturer's hold-down connector values will be allowed, and 3x members are required at the bottom sill plate. Two 2x members joined together may be used in earthquake repair work. The City of Los Angeles now requires that plate washers be used for sill plate anchor bolts, instead of split washers, to help force the bolt to go into double curvature, which usually is associated with a higher level of force (yield mode IV). Larger washers also allow for more effective tightening of the bolts, and more friction is developed in the connection. Finally, the use of a larger washer prevents the nut from crushing the wood around the hole and pulling through the sill plate.

CHAPTER III

EXPERIMENTAL TESTING PROGRAM

The experimental testing program performed in this research study was composed of two major tasks. The first task involved testing various sill plate-to-concrete connections in order to evaluate the accuracy of the NDS design assumptions for wood-to-concrete connections. The effects of using reinforcing clamps to confine the sill plate was also determined. In the second task, full-scale plywood shear walls were tested to determine their response performance and evaluate the effects of using reinforcing clamps in critical connection locations. In both test programs, quasi-static reversed cyclic loading was used.

3.1 Task 1: Sill Plate Component Testing

The sill plate component testing program was developed to evaluate the design strength equations from the National Design Specifications for applicability to wood-to-concrete connections. Previous tests (Stromatt 1996) have verified the accuracy of the yield limit equations for a single thickness main wood member and a double thickness side wood member. According to the NDS, this configuration should accurately model the strength of a wood-to-concrete connection. A description of the test specimens, reinforcing clamps, loading history and data acquisition follows.

3.1.1 Specimen Description

The series of tests performed in this task involved sill plate connections using 2x4, 2x6, and 3x4 lumber with and without reinforcing clamps. The lumber species of the sill plates was Southern Pine (Specific Gravity = 0.55). These members were connected to a reinforced concrete foundation with ASTM A36 1/2" and 3/4" diameter bolts. A description of the various specimens is shown in Table 3-1.

TABLE 3-1. Experimental Testing Program for Component Tests

Specimen #	Sill Plate Size	Bolt Diameter (in)	Reinforcing Clamp	Calculated Yield Mode (NDS 1991)
1	2x4	1/2		III
2		1/2	X	III
3		3/4		III
4		3/4	X	III
5	3x4	3/4		III
6		3/4	X	III
7	2x6	1/2		III
8		1/2	X	III
9		3/4		III
10		3/4	X	III

Ten (10) tests were performed with several combinations of lumber size, bolt diameter, and sill plate confinement. Since only one sample per combination was performed, no meaningful statistical analysis could be performed. However, it was hypothesized that general trends would be apparent, and could be verified with future testing. From the yield mode equations in NDS (1991), the 5% diameter offset strengths are found for these members, using ASTM A36 steel and the assumption that the thickness of the main member, t_m, is twice the thickness of the side member, t_s. The 5% diameter offset strength represents an estimate of the yield strength of the connection and

is achieved by removing the factor of safety present in the denominator of the yield mode equations. Table 3-2 presents the calculated allowable design and 5% diameter offset strengths of the connections in this testing program.

TABLE 3-2. Expected Performance of Wood-to-Concrete Connection Specimens (From NDS 1991)

Sill Plate Thickness (inches)	Bolt Diameter (inches)	Allowable Design Load (kips)	Governing Yield Mode	5% ϕ Offset (kips)
1.5	1/2	1.05	IIIs	2.11
(2x4, 2x6)	3/4	2.04	IIIs	4.08
2.5	1/2	1.20	IV	2.40
(3x4)	3/4	2.51	IIIs	5.02

3.1.2 Reinforcing Clamps

In order to confine the sill plate and deter splitting (non-ductile behavior), special reinforcing clamps are used on some specimens* . The clamps are constructed of 16 gauge galvanized steel, and contain an oversized or slotted hole through which the bolt passes. Helically threaded nails are driven into the sides of the wood member so that the side walls of the clamp are restrained from slipping and deflecting outward if the member splits. A detail of the clamps is shown in Fig. 3.1.

The reinforcing clamps can be used to provide both enhanced strength and ductility to the connection. If a large diameter bolt is used, the clamp will confine the sill plate so that the board will retain strength, even if splitting occurs. With smaller diameter anchor bolts, the connection can be changed from a mode III response (single curvature) to a mode IV response (double curvature) by placing a heavy washer above the

* These clamps are the intellectual property of The Texas A&M University System.

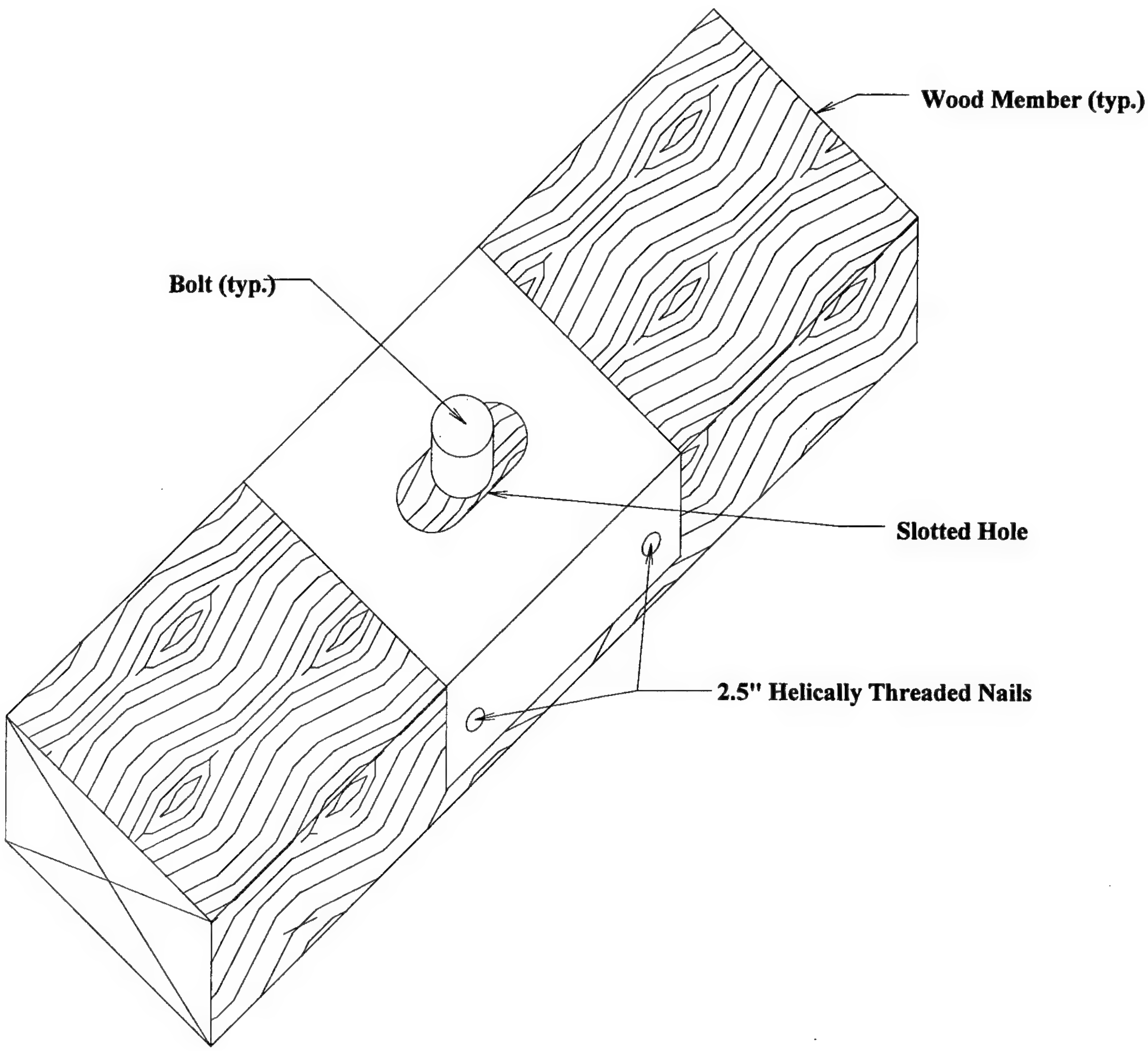


FIG. 3.1. Detail of Typical Confining Clamp

clamp. This will usually result in a higher yield strength. If splitting occurs, the clamp provides transverse resistance to the wood member and ensures that sufficient lateral strength and energy dissipation capacity exists in the connection. In summary, the clamp can be used to ensure the failure does not occur in the wood member, but in low-cycle fatigue of the anchor bolt.

In addition to lateral shear resistance, the reinforcing clamps can provide resistance to out-of-plane tensile loads. These loads act in a direction perpendicular to the grain of the wood, which is the weakest plane in wood, and out-of-plane tensile loads can contribute to brittle splitting along the grain of the sill plate. By preventing this failure, the connection can retain strength and deformation capacity.

3.1.3 Test Setup

The experimental test setup for the sill plate-to-concrete connection testing is shown in Fig. 3.2. Anchor bolts were embedded in a reinforced concrete base, which was 1.0 foot deep 1.5 feet wide, and 10 feet long. The lateral shear force was applied to the specimens by means of a 22 kip servo-controlled hydraulic actuator. Relative displacements were measured by means of a sonic displacement transducer, and the applied load was measured from a load cell on the actuator.

3.1.4 Loading History and Data Acquisition

In order to compare the results of this testing program to previous results dealing with wood connections, the same procedure used in previous wood connection tests at Texas A&M University was followed (Stromatt 1996). Incremental quasi-static reversed cyclic loading was used to identify the response behavior of the connection at the elastic, yield, and ultimate limit states. Displacement control loading was used by increasing the actuator stroke incrementally, with two cycles at each stroke level. The loading was applied at a rate of 30 seconds per cycle. Displacement amplitudes began at 0.25" and continued with 0.25" increments until the connection failed or the limit of the actuator was achieved. A PC based data acquisition system was used in order to record the response data at a rate of 2 samples per second.

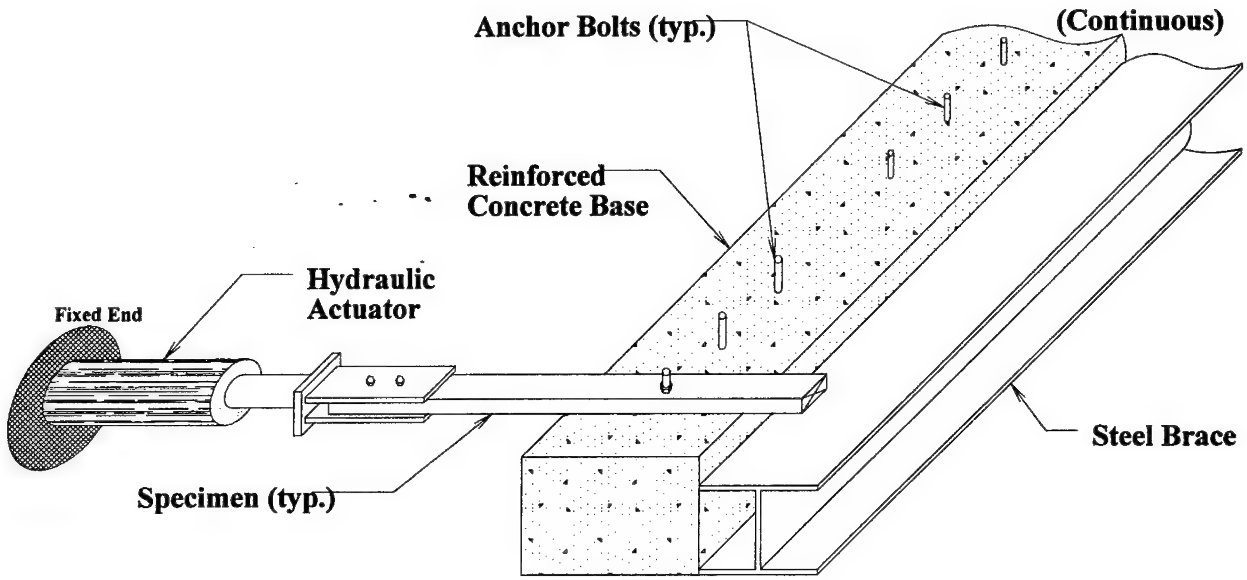


FIG. 3.2. Experimental Test Setup -- Task 1: Sill Plate Component Tests

3.2 Task 2: Full-Scale Plywood Shear Wall Testing

The full-scale plywood shear wall tests were designed to determine the effects of confining devices on the overall performance of shear wall specimens. The shear walls were designed in accordance with UBC (1994) guidelines and reinforced with the same type of clamps used in the component tests (Task 1). There were three tests performed in this task: (i) an unreinforced shear wall; (ii) a shear wall reinforced with confining clamps on the bolted connections; and, (iii) the reinforced wall, repaired after testing. A description of the test parameters and setups follow.

3.2.1 Model Design and Construction

The specimens in this testing program were designed to be similar to the shear wall specimens in the APA sponsored tests at the University of California - Irvine. The wall specimens had dimensions of 8' x 8', with 2x4 studs spaced at 16" o/c. Plywood sheathing with a thickness of 15/32" was used. The wall had four connections to the concrete base. Two connections were hold-down connections while the other two were anchor bolt (shear) connections. The tests in this study differed from the UC Irvine tests in that the desired response was failure of the bolted connections. A 2x4 member was used for the sill plate, and the wall was constructed to resist a high level of shear. The design procedure for the walls follows.

<u>Step 1 -- Capacity of Bolts in Sill Plate:</u> The sill plate connection strength can be found by evaluating the equations found in part VIII of the NDS (1991). These equations evaluate the governing yield strength associated with deformation modes I-IV. For

common dimension lumber and bolt sizes in single shear connections, these values are summarized in NDS (1991) Table 8.2A. For wood-to-concrete connections, the thickness of the main member, t_m, should be taken as twice the thickness of the side member, t_s. Therefore,

$t_s = 1.5"$ (2x4 Sill Plate)
$t_m = 3"$ ($t_m = 2t_s$)
$G = 0.55$ (Specific Gravity of Southern Pine)
$D = 1/2", 3/4"$ (Bolt Diameters)

From NDS (1991) Table 8.2A, the nominal strength for load parallel to the grain, $Z_{\|}$, is 660 lbs for the 1/2" bolts, and 1270 lbs for the 3/4" bolts. Both connection types correspond to mode III deformation response (single curvature of the bolt). The allowable design strength, obtained by multiplying the nominal design value by a load duration factor (C_D) of 1.6 for earthquake loads, is:

$$1.6[2(660\text{ lbs}) + 2(1270\text{ lbs})] = \mathbf{6176\ lbs}$$

All other applicable factors (C_M, C_t, C_G, and C_Δ) are unity.

The next step is to check the bearing capacity of the bolts in concrete. From UBC Table 26-E, the allowable shear capacities of 1/2" and 3/4" bolts in concrete are 2000 lbs and 3560 lbs, respectively. The total resistance is therefore:

$$2(2000) + 2(3560) = \mathbf{11120\ lbs}$$

Since the allowable design strength of the bolt in the wood plate is lower than the allowable capacity of the bolts in the concrete, the design value for the sill plate connection is 6176 lbs.

The 5% diameter offset strength of the connection is found by removing the factor of safety from the denominator of the NDS yield mode equations. The 5% diameter offset strength represents the yield strength of the connection. Since both of the connections in the shear wall specimen correspond to yield mode III behavior, the 5% offset strength is found to be:

$$3.2[(2(660 \text{ lbs}) + 2(1270 \text{ lbs})] = \mathbf{12352\ lbs}$$

The NDS (1991) allowable strength design, bolted connections have a high factor of safety between the design and ultimate strength. By removing the safety factor in the design equations, the 5% diameter offset load is determined. However, this is still not the maximum load which may be achieved. Since failure of the connection is critical to the tests, the expected ultimate load was calculated as described below. From the most recent experimental tests on the bolted wood connections (Stromatt 1996), a factor of safety of approximately 2.84 was observed for unconfined wood members. Since these tests used a main member thickness of 3", side member thickness of 1.5", and 1/2" and 7/8" anchor bolts, it is reasonable to expect a similar factor of safety in these calculations. Therefore, the maximum strength of this connection is found by multiplying the design load by the factor of safety:

$$2.84(6196 \text{ lbs}) = \mathbf{17537\ lbs\ (17.5\ kips)}$$

<u>Step 2 -- Shear Wall Design:</u> The design lateral resistance of the sill plate was found to be 6176 lbs. Dividing this shear by the length of the wall (8') gives a unit shear of 772 lbs/ft. Using UBC Table 25-K-1, with 15/32" plywood and size 10d common nails, the nailing schedule was found which produces a wall capable of resisting at least

that level of shear force. The lateral shear resistance in the wall also has an inherent factor of safety. Shear wall tests at the University of California, Irvine estimate that the maximum strength of the wall is 2.6 times the allowable design strength. Therefore, this value was assumed to be applicable to the wall specimen in this study due to the similarity in the wall section, materials, and testing methods. Table 3-3 shows the available nailing schedules.

TABLE 3-3. Shear Wall Design Table (UBC 1994)

Nail Spacing at Panel Edges (in.)	Design Unit Shear (lb/ft)	Total Shear Resistance (k)	Total Shear including F.S. = 2.6
4	510	4.08	10.6
3	665	5.32	13.8
2	**870**	**6.96**	**18.1**

Therefore, in order to force the failure mechanism into the bolted connections, 2" o/c spacing on the perimeter of the plywood panel was selected. According to the UBC (1994), nails should be spaced a maximum of 12" o/c along intermediate supports, but in this test program the spacing was decreased to 6" o/c in order to provide additional strength to the wall.

The final failure mode to check is the compression and tension forces on the framing members. From equilibrium, since the wall specimens are square, the estimated tension or compression forces on the end studs of the wall is equal to the applied force (17.5 kips). This is a conservative upper limit as some of the force will be carried by the anchor bolts in the center, and deformation in the sill plate will give an area of compression rather than a point, which will lower the moment arm. The selected hold-

down connector was the Simpson Strong Tie Co. HD5A, which has a tabulated ultimate strength 20.8 kips. Under a service load of 3.705 kips, this prefabricated metal bracket should deflect 0.0675 inches. This hold-down connection also includes the bolted connection to the boundary stud of the wall, two 3/4" bolts passing through the double 2x4 end post and the steel tie-down. From NDS table 8.2B, for a 3" thick Southern Pine member, 1/4" ASTM A36 side plate, and 3/4" bolts, the allowable design strength is 4.99 kips for the connection. With the 2.8 factor of safety for bolted wood connections, the maximum load expected is 14.17 kips.

According to the UBC (1994), the center stud must be at least a 3x member, and standard construction requires a double stud at the ends and a double top plate. Since 3x members are typically not available in the southern region, a double stud was also used in the center of the wall panel. Table 3-4 summarizes the maximum strengths expected from the wall specimen.

TABLE 3-4. Plywood Shear Wall Test Specimen Design Summary

Failure Mode	Design Value (k)	Ultimate Value (k)
Sill Plate Bolts	6.20	17.5
Tension Member Bolts	4.99	14.2
Shear Wall	6.96	18.1

The required depth of the concrete base needed to be at least 11 inches so that the embedment requirements for the hold-down bolts were met. In addition, the holes in the laboratory strong floor are spaced at three feet on center. Therefore, the reinforced concrete base was 1 foot deep, 1 foot wide, and ten feet long. There were two #5

reinforcing bars top and bottom, with #3 stirrups at three to six inch spacing. A detail of the specimen setup is depicted in Fig. 3.3.

3.2.2 Reinforcing Clamps

In order to confine the wood members and prevent splitting (non-ductile behavior) in critical locations, special reinforcing clamps were used in the second wall specimen. These clamps were identical to those used in the component testing, and performed in a similar fashion. The clamps are shown in Fig. 3.4, along with their positioning in the second model. Additional metal brackets were attached in the corners of the wall to further connect the sill plate to the tension post, limiting uplift deformations. These additional clamps are also shown in Fig. 3.4.

3.2.3 Test Setup

The testing of the wall specimens used a hydraulic actuator attached to the top of the wall with a steel W-section, in order to distribute the applied lateral load along the length of the wall. The wall was attached to a concrete foundation, and bolted into the strong floor. The wall was restrained from out-of-plane buckling by braces at both ends of the wall. Displacement measurements were taken at seven places: (i) two sonic displacement transducers were used to measure the lateral deflection at the top and bottom of the wall; (ii) three linear potentiometers were used to measure the uplift at the wall ends as well as any sliding of the base. These instrument locations are prescribed by ASTM E564-76; and (iii) two clinometers were attached to the plywood sheathing to

measure the angle of rotation. The load cell on the actuator was used to measure the applied lateral load. Fig. 3.5 shows the test setup and the attached instrumentation.

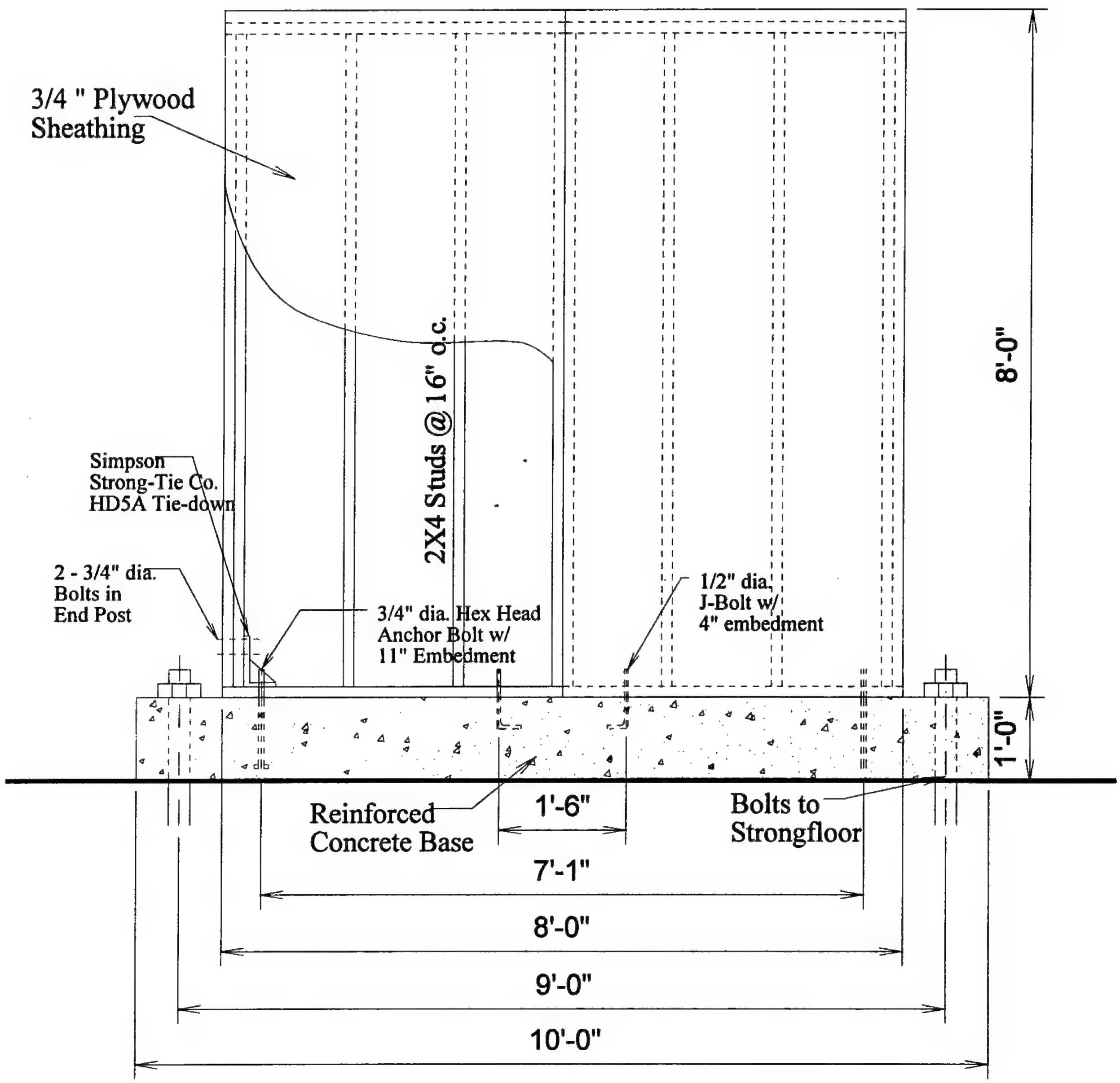
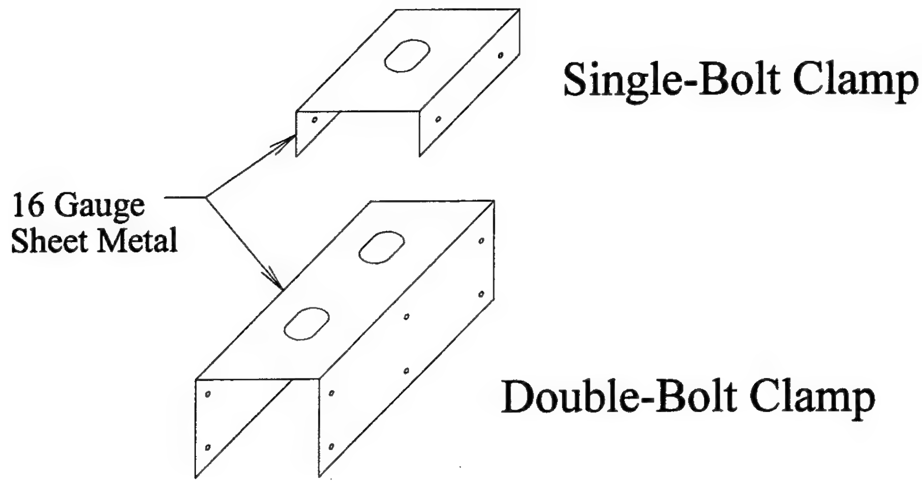


FIG. 3.3. Specimen Construction -- Task 2: Full Scale Plywood Shear Wall Testing



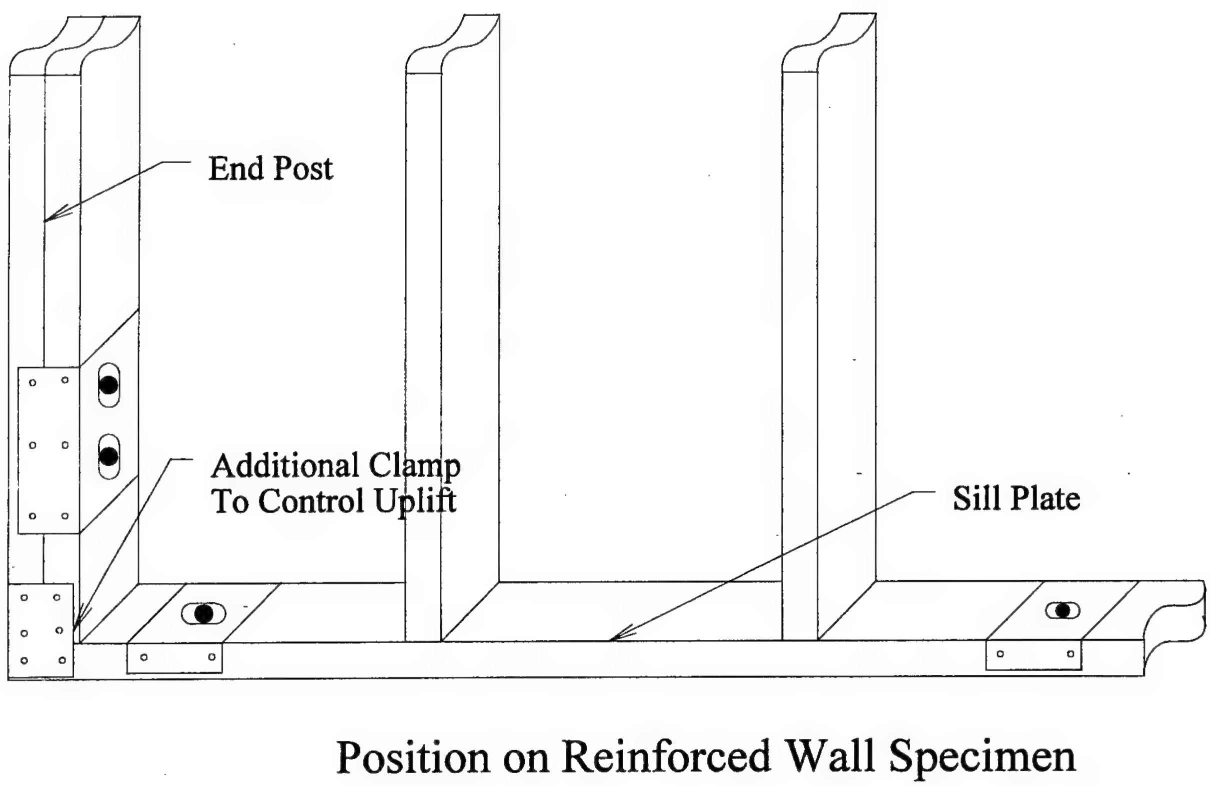


FIG. 3.4. Confining Clamps -- Task 2: Full Scale Plywood Shear Wall Testing

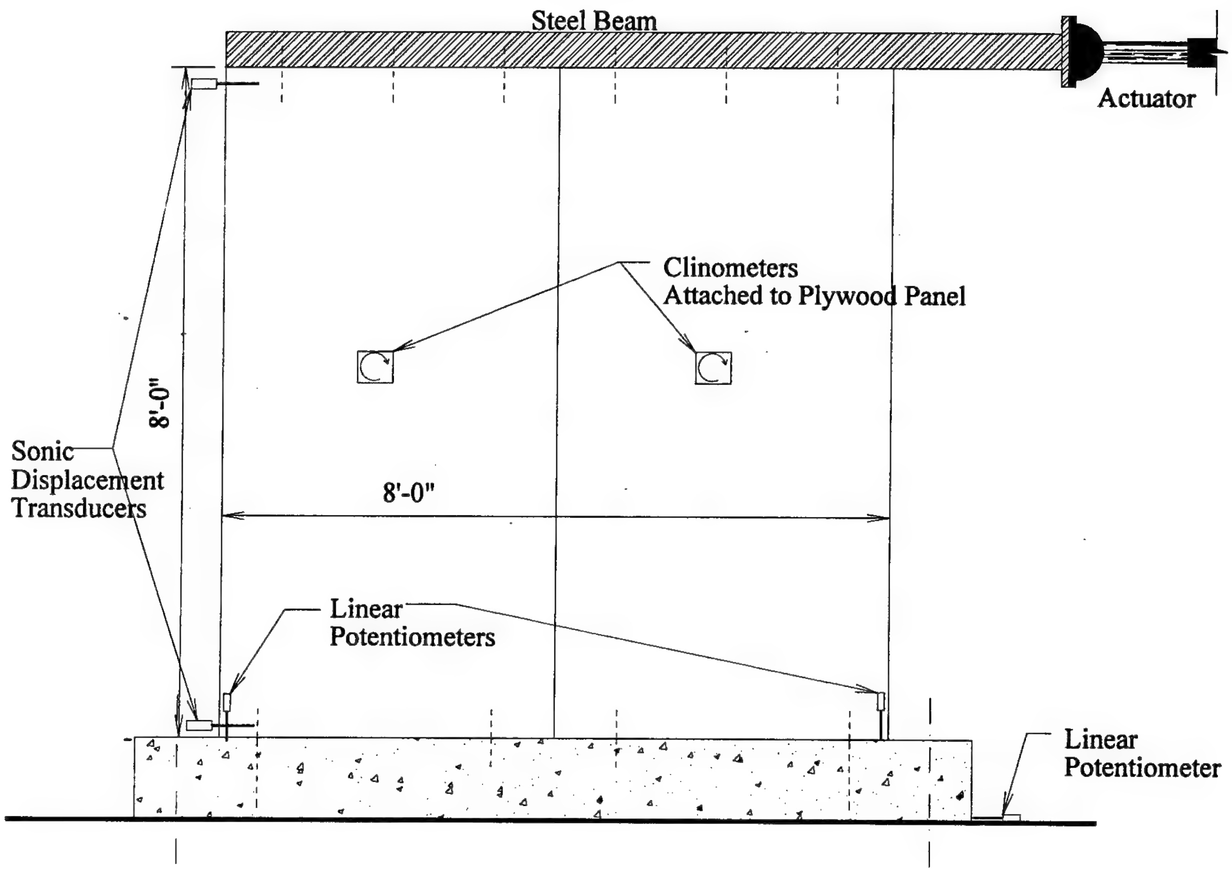


FIG. 3.5. Experimental Test Setup: Task 2: Full Scale Plywood Shear Wall Testing

3.2.4 Loading History and Data Acquisition

Since there is no standard for cyclic loading of structural subassemblies, quasi-static reversed cyclic loading was used in this task. Incremental displacement control loading was used, with two cycles at each displacement level. The rate of loading of the actuator was set to 30 seconds per cycle. Increasing displacement amplitudes of 0.125", 0.25", 3/8", 1/2", 5/8", 3/4", 1", 1 1/4", 1 1/2", 1 3/4", 2", 2 1/4", 2 1/2", 2 3/4", and 3" were used. The loading history is shown in FIG. 3.6. A PC based data acquisition system was used to record the data at a rate of 4 samples per second.

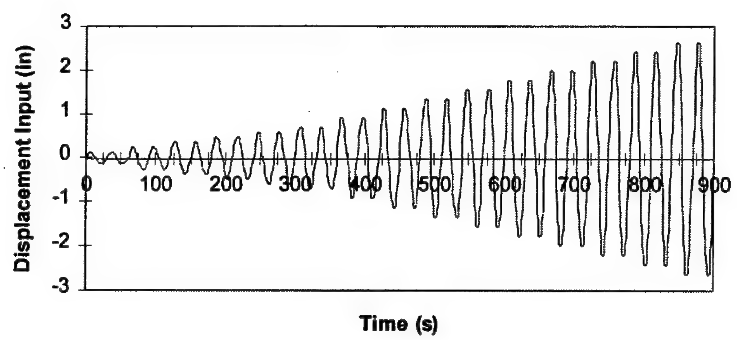


FIG. 3.6. Displacement Loading History for Full-Scale Plywood Shear wall Testing

CHAPTER IV

EXPERIMENTAL RESULTS

Experimental results for the sill plate-to-concrete foundation component testing and for the full-scale plywood shear wall tests are presented in the following sections.

4.1 Task 1: Sill Plate Component Tests

The performance of the wood-to-concrete connections evaluated in this testing program is discussed in the following sections. The applicability of the NDS design assumptions for this type of connection is evaluated, and then the effect of the confining clamps on the strength, ductility and energy absorption capacity of the connection is examined. The response behavior of the specimens in this testing program are included in Appendix B, Figs. B1 through B15.

4.1.1 Yield Mode Comparison

The response behavior in all testing performed in this task indicated that the initial yield response was governed by mode III behavior, or yielding of the bolt accompanied by local crushing of the wood fibers. The bolts yielded at about 1/2" below the surface of the concrete base. However, the confined 2x4 member using a 1/2" diameter bolt (Specimen 2) did experience some deformation response resembling double curvature, or mode IV behavior. The primary yield point was slightly within the concrete base, as was seen in the other tests. The secondary yield point was at the base of the nut. The tests show results very close to those predicted by the NDS equations. The concrete base did

experience some minor spalling at the point of bearing for the bolt. Therefore it was confirmed that the NDS double main member assumption is reasonable, and the yield strength of the connection is not significantly affected by the stiffness of the concrete. Since the concrete experienced some degree of local crushing near the yield point of the bolt, it behaved in a similar fashion to a wood member. Table 4-1 summarizes the results of the tests.

On the test of the confined 2x6 with a 3/4" bolt (Test #10), the nut was improperly tightened and came off the bolt, causing the 2x6 to move off of the connection. This action was caused by an anchor bolt which was sunk too far into the base, and would probably not occur in actual construction. For this reason the test was considered void, and nine tests were available for analysis.

TABLE 4-1. Response Behavior of Sill Plate Specimens

Test #	Initial Stiffness (k/in) + / -	Post-Yield Stiffness (k/in) + / -	5% Diameter Offset Yield Strength (kips)			Yield Mode	
			NDS	TEST (+/-)	AVE. RATIO	NDS	TEST
1	15 / 17.5	-0.44 / -0.80	2.11	2.5 / 2.8	1.25	III	III
2	32 / 36	0 / -1.0	2.11 2.40	4.2 / 3.0	1.71 1.50	III	III / IV
3	36 / 38	0 / -1.0	4.08	4.4 / 4.7	1.11	III	III
4	45 / 53	0.53 / 0.80	4.08	4.0 / 4.4	1.03	III	III
5	44 / 15	-0.09 / -0.36	5.02	4.8 / 4.1	0.886	III	III
6	44 / 27	1.09 / -0.50	5.02	4.7 / 4.2	0.886	III	III
7	10 / 26	0.06 / -0.17	2.11	2.3 / 2.8	1.21	III	III
8	44 / 12	2.29 / 1.63	2.11	2.2 / 2.3	1.07	III	III
9	7.3 / 32	0.66 / 0.80	4.08	4.4 / 6.4	1.32	III	III
10	VOID TEST						

Therefore, the NDS yield mode equations provide an reasonable description of the yield behavior of wood-to-concrete bolted connections. It is observed that generally the NDS predictions are slightly conservative, but close enough to the observed behavior to justify the assumptions made for wood-to-concrete connections.

4.1.2 Failure Mode Comparison

The experimental testing indicated a notable feature of the ultimate failure mode of wood-to-concrete connections. In a wood-to-wood connection, the bolt can move throughout the whole main member, and the local crushing provides more flexibility. In the testing, the bolts were held more rigidly by the concrete in a wood-to-concrete connection. Since the bolts are not allowed to shift in the main concrete member, the deformation is localized at the yield point. This leads to a higher incidence of low-cycle fatigue and bolt rupture. The performance at failure is shown in Table 4-2.

TABLE 4-2. Performance at Failure of Sill Plate Specimens

Test #	Sill Plate	Bolt ϕ (in)	Clamp	Maximum Load (k)	Disp. @ Failure (in)	Energy Absorbed (kip-in)
1	2x4	1/2		3.24	Split @ 1.25"	14.79
2		1/2	X	4.77	Bolt Fracture @ 1.25"	18.35
3		3/4		5.80	Split @ 0.75"	15.00
4		3/4	X	6.37	Bolt Fracture @ 1.5"	55.03
5	3x4	3/4		6.61	Bolt Fracture @ 1.5"	63.16
6		3/4	X	7.56	Bolt Fracture @ 1.5"	69.74
7	2x6	1/2		5.11	Bolt Fracture @ 1.0"	21.91
8		1/2	X	4.76	Bolt Fracture @ 0.75"	17.20
9		3/4		5.95	Bolt Fracture @ 1.5"	52.00

4.1.3 Influence of Confinement

The response behaviors of 2x4 specimens with a 1/2" diameter anchor bolt (Tests #1 and #2) are depicted in Figs. B1 and B2. A comparison of the load-displacement relationships show a 50% increase in the ultimate strength of the confined connection, which is partly due to the clamp forcing the bolt to deform in double curvature (mode IV behavior). The initial stiffness of the connection is twice as great with the confining clamp. Fig. B3 shows the energy absorbed by each connection, and the confined connection dissipates 24% more energy than the unconfined.

The load-displacement relationships for the 2x4 members with 3/4" diameter bolts (Tests #3 and #4) are shown in Fig. B4 and B5. It can be observed that the unconfined specimen split at a small displacement (≈ 0.5"), with subsequent loss of strength. The confined specimen also split but was able to withstand twice the deformation before the bolt fractured. Both members reached the same level of strength, but the increased ductility provided by the confining clamp allowed the confined member to absorb 266% more energy, see Fig. B6. Similar to tests #1 and #2, tests #3 and #4 showed an increase in the initial stiffness of the connection. Tests #3 and #4 demonstrate the importance of the confining clamp on bolted wood connections where splitting governs as a failure mode.

The load-displacement relationship for the 3x4 specimens with 3/4" diameter bolts (Tests #5 and #6) are shown in Figs. B7 and B8. The performance of the specimen with and without confinement is almost identical. The confined specimen does, however, maintain a higher level of strength at high levels of displacements, which is probably due

to the clamp preventing the bolt from damaging the top of the member, and go into double curvature. Because of this behavior, the post-yield stiffness is much higher in the confined specimens. Both specimens failed by fracturing of the bolt at about 1.5" relative deformation. It is shown in Fig. B9 that the additional strength at high displacements allows for a 10% increase in energy dissipation capacity for the confined specimen (Test #6).

In the 2x6 members with 1/2" diameter bolts (Tests #7 and #8), Figs. B10 through B12, similar results are shown. In this case, the clamp does not seem to have an effect on the overall response. In fact, the unconfined member shows a higher ultimate strength and energy dissipation. No splitting was observed in any of the 2x6 members, even with the 3/4" diameter bolt (Figs. B13 through B15). When splitting is not the critical failure mode, the confining clamp generally does not result in a significant improvement in performance, but it does not hinder the performance either. Further tests should be performed in order to determine whether this is representative of actual 2x6 behavior. Wood members have a considerable amount of variability, and all sections used in this test were cut from one piece of lumber.

4.2 Task 2: Full-Scale Plywood Shear Wall Tests

The performance of the original and reinforced shear walls is evaluated in terms of: (i) initial stiffness; (ii) ultimate strength-deformation capacity and performance; and (iii) energy absorption. These results will discuss both the quantitative results obtained

from the data acquisition and analysis, as well as the qualitative observations of damage and performance.

Each area of evaluation is further broken down into the components of deflection as shown in Fig. 1.1. The sliding deflection was taken from the displacement measurements at the bottom of the wall. During testing, it was determined that the wall did not perform as a single unit. The two plywood panels deflected independently (See Fig. 4.1). This was due to the built-up center stud. The two members in the center slid in relation to each other, and did not perform like a 3x4 member would. Because of this action, the overturning deflection was twice that of a square wall, and was taken as being the twice as the uplift at one end of the wall, since the independent panels had a 1:2 aspect ratio. The shear and bending deflections are the remaining contributions to the total deflection. This component is primarily shear, or racking deflection, as the bending component is relatively small for a wall with a 1:2 aspect ratio.

On the first series of tests, the PC-based data acquisition system malfunctioned during the tests, and much of the data was lost. Fig. C1 in Appendix C shows all of the data recorded from the wall tests. Although much of the data was lost, many comparisons can be made between the performance of the two walls. Where data is missing, the results are interpolated from the trends seen in the second test. Fig. C2 shows the interpolation of data. The major results are summarized in Table 4-3.

TABLE 4-3 -- Summary of Task 2: Full Scale Plywood Shear Wall Testing

Specimen	Shear Stiffness (G') (k/in)	Maximum Load Achieved (kips)	Energy Dissipation Ratio
Unreinforced	13.5	8.14	1.0
Reinforced	18.75	9.19	1.26
Repaired	9.93	7.72	1.30

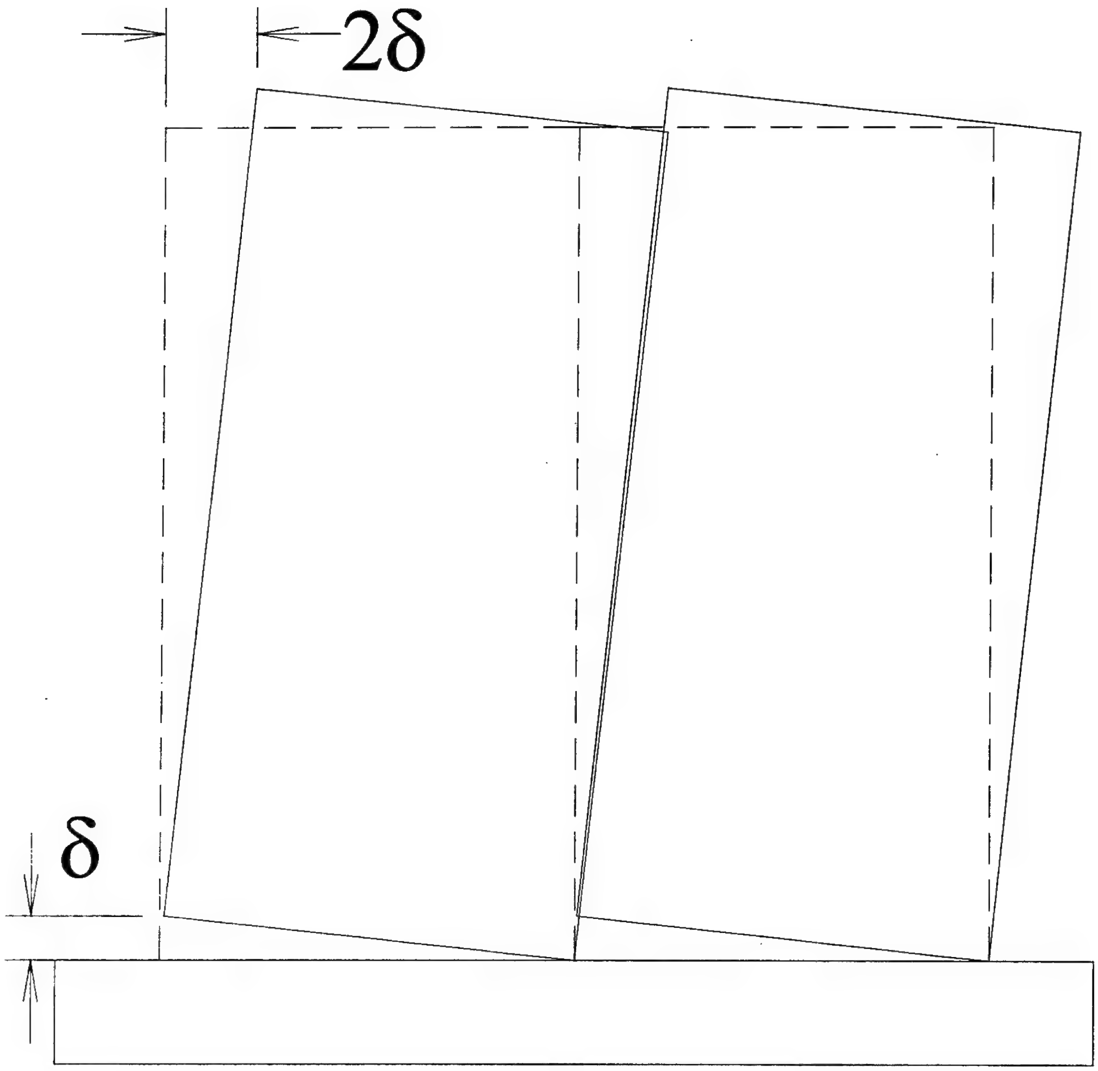


FIG. 4.1. Actual Overturning Deflection in Shear Wall Testing Program

4.2.1 Initial Stiffness

The load-deflection relationships of the walls at the initial small displacement amplitudes are compared in Figs. C3 through C5 in Appendix C. Fig. C3 shows the sum of all components, or the total displacements. From that relationship, initial stiffnesses can be found. In ASTM standard E 564, the method of computing shear stiffness, G', is given as:

$$G' = \frac{P}{d} \times \frac{a}{b}$$

P is the concentrated load at the top of the wall, d is the total deflection, a is the height of the wall, and b is the length of the wall. From this equation, the stiffness of the first wall is found to be 13.5 k/in, and the stiffness of the reinforced wall is found to be 18.75 k/in. This is a forty percent increase in initial lateral stiffness. By analyzing the components of the lateral displacement, it is seen that the difference in stiffness comes from a major reduction in the overturning displacements (see Fig. C4). When these displacements are subtracted, the shear and bending stiffness (Fig. C5) appears to be nearly identical. The displacements due to sliding displacements are not graphed, as they are insignificant for initial loading.

These results show the effectiveness of the reinforcing clamps in improving the initial stiffness of the wall. Since this stiffness is associated with elastic force, it represents the performance of the wall in service, or during elastic response. In these conditions, minimizing deflection is desirable to preserve the integrity of the wall coverings and the overall stability of the structure.

4.2.2 Large Deflections

The load-deflection curves for the walls at large deflections (near capacity of the specimens) are included in Appendix C, Figs. C6 through C9. From the total displacement, Fig. C6, it can be observed that the ultimate load achieved is higher for the reinforced wall then for the unreinforced wall. The hysteretic curve for the reinforced wall also shows less "pinching" or slip, so it indicates a higher amount of energy absorption.

For the overturning component (see Fig. C7), the reinforced wall shows significantly less deflection, and also more energy absorption. The same is true for the sliding component, (see Fig. C8). These are the bolted connections, and improvement in the performance of these connections is the apparent cause of the overall improvement in the wall performance. The shear/bending component (Fig. C9) is very similar for both walls, although it shows less strength degradation in higher cycles. This is probably due to the fasteners at the bottom of the wall maintaining their integrity, since there is much less uplift.

The lateral load-displacement backbone curves for the reinforced wall can easily be found, since all data is present. It is assumed that performance of the unreinforced wall will be similar, so a curve can be interpolated for it as well. The backbone curves are given in Fig. C2.

In the first test of the unreinforced wall, the sill plate showed a great deal of warping as the tension post lifted up, pulling the plywood panel up. This applies a great deal of force into the nails attached to the sill plate, cupping the plate and pulling the nails

out. A crack formed in the end of the sill plate, extending through the hold-down bolt hole and to the end of the member. This behavior was not observed in the reinforced wall. The bottom plate remained intact, as the uplift in the tension post was reduced considerably, due to the reinforcing clamp group. There was still withdrawal and fracturing of the nails, but that apparently was due to shear and bending deformations rather than uplift of the end post. After the test, the sill plate was significantly less damaged in the reinforced wall, so repairing the wall would be an easier task than having to replace the sill plate.

4.2.3 Energy Absorption

Fig. 4.1 shows the increase in energy dissipation from the unreinforced to the reinforced wall. The energy dissipation calculations were taken from the final portions of both tests, where data was available for both walls. By using the same separation technique as before, the energy absorbed by each component of deflection was computed.

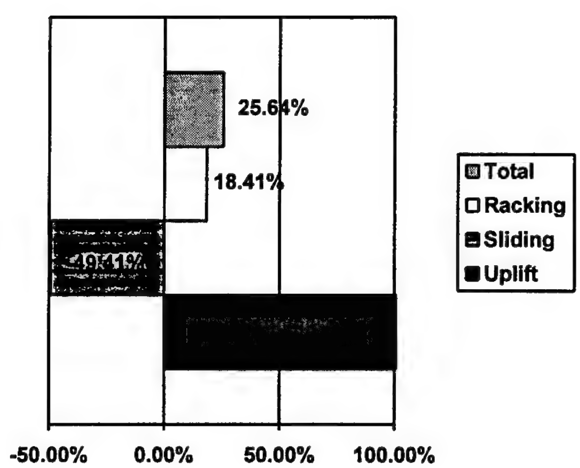


Fig. 4.2. Increase in Energy Dissipation for Reinforced Wall

It can be observed that a 25% increase in the total energy dissipation results from the inclusion of the confining clamps on the bolted connections. This can be attributed to a 100% increase in the energy absorbed by the overturning deformation and an 18% increase in the energy absorbed by shear and bending deformations. The decrease in uplift apparently decreased the stress on the bottom nails, so they could more effectively resist shear force. The graph also indicates a 50% decrease in the energy absorbed by sliding deformation, but since the sliding of the reinforced wall was practically eliminated, this component could not absorb energy.

As mentioned previously, in both tests each 2x4 in the center stud slid vertically in relation to the other. This stud was nailed with the UBC (1994) required 16d at 24 inches o/c. The slipping of these connectors could also have helped absorb energy, and increasing the nailing of this type of connection may increase that performance.

4.2.4 Wall Repair

After performing the tests on the reinforced wall, the reinforced wall was repaired by replacing nails which had ruptured or pulled out with 2" helically threaded nails. The wall was then tested, again using quasi-static reversed loading. Appendix C contains the data from these tests. The total load-displacement response, initial stiffness, and energy dissipation are shown in Figs. C10 - C12, respectively.

The repaired wall showed a 16% drop in maximum strength, a 47% drop in initial stiffness, but about the same level of energy absorption as compared to the undamaged reinforced wall. The post-yield stiffness recovered about halfway from the final damage

state of the reinforced wall. The repaired wall, therefore, showed similar performance to the unreinforced wall. A comparison of the backbone curves is shown in Fig. C13.

The energy absorption similarity was probably due to the decreased stiffness of the repaired wall. The wall had slightly higher displacements per cycle, and because it did not rebound as effectively as the undamaged wall, it did not return energy as effectively.

4.2.5 Specimen Accuracy

The specimens in this research differ from a plywood shear wall in residential or commercial use in two important areas: (i) there are no interior or exterior coverings; and (ii) there is no significant dead load from a roof. The inclusion of gypsum wallboard should not affect the performance of this plywood shear wall because gypsum degrades quickly under cyclic loading. Exterior coverings have been found by the APA to not have a significant impact on the strength of a shear wall, so they are not necessary.

With respect to dead load, because of the variation in residential and commercial structures, it is hard to estimate the dead loads present in a wall. A typical number (Fausett & Muller 1993) for roof and ceiling dead loads is 184 pounds per linear foot (plf). With the inclusion of typical attic storage (100 plf) the value becomes 284 plf. In the plywood shear wall model used in this study, that value would add only a 1136 lb additional force on the end post. This would improve the performance of the wall at small deflections by holding the end post down, but at large deflections typically experienced in an earthquake, the force would be insignificant. In these tests a maximum

load between 8000 and 9000 pounds was achieved. Because of the 1:2 aspect ratio, this means a 16 to 18 kip uplift force in the end post, so the dead load would represent only a six to seven percent decrease in the uplift force. Although larger structures and structures with multiple floors would have a larger dead load, the additional weight would also act to increase the seismic inertial force, and again the downward force would be small. Additionally, shear walls are usually located at the corners of the structure, where the dead load is smallest.

CHAPTER V

CONCLUSIONS

5.1 Task 1: Sill Plate Component Tests

The experimental study of bolted wood-to-concrete connections provided insight into the performance of these connections during quasi-static reversed loading. The standard design assumptions made in the NDS were validated, and the performance of confining clamps was investigated. It can be summarized that:

(1) The design assumption from the NDS, to take the thickness of the main member (t_m) be taken as twice the thickness of the side member (t_s), provides an accurate calculation of the 5% offset strength of a wood-to-concrete connection. The higher stiffness of concrete did not seem to affect which yield mode was developed or the yield strength, as it is subject to spalling where a wood member would undergo local crushing.

(2) The ultimate failure mode of a 2x4 sill plate connection is splitting of the lumber, and for larger wood members, low-cycle fatigue of the anchor bolt. The use of a confining clamp on 2x4 members prevents the sudden loss of strength capacity if splitting occurs, and allows the connection to absorb much more energy before failure.

(3) The use of a confining clamp on a 2x4 with a 1/2" bolt provided a significant increase in strength and initial stiffness. The clamp forces the bolt to deform in double curvature, which is a higher strength yield mode. With the increase in strength and deformation capability there was a corresponding increase in energy dissipation.

(4) The clamp on the 3x4 resulted in an increase in post-yield stiffness. This was due to the clamp forcing the bolt into double curvature at large displacements. In the unconfined member, the bolt was able to crush the top of the wood, and the damage resulted in a progressive loss of strength. The increase in post-yield stiffness of the confined specimen resulted in an increase of energy dissipation.

(5) Where splitting was not the failure mode, the clamp did not have a drastic improvement, but the performance of the connection was not hindered by the clamp. Since splitting has been observed in post-earthquake inspection, the clamps would be a wise inclusion into construction practice, as an easy way to prevent splitting failures.

5.2 Task 2: Full-Scale Plywood Shear Wall Tests

The experimental study of full-scale plywood shear walls provided insight into the performance of wood frame walls and the performance of their bolted connections. In addition, the effects of providing reinforcing clamps on these connections and the overall improvement in the performance of the shear wall was observed. It can be summarized that:

(1) The connection between the wall end post and the sill plate is a critical connection to the overall response behavior of the wall. The uplift deformation in this connection directly contributes to the overall deflection of the wall by a factor of two. This deformation was the largest component of deflection. By minimizing this uplift

deformation, the initial lateral stiffness, overall performance, and energy dissipation capability of the wall can be improved.

(2) Use of a built-up double 2x4 stud where panels adjoin is not sufficient to ensure the wall functions as a unit. Since the wall panels act separately, the uplift force is doubled, and overturning deflection is twice the uplift. This significantly hinders wall behavior, as the overturning deflection is the most significant component of deflection.

(3) The inclusion of confining clamps in the critical bolted connections of the shear wall increases the energy dissipation capability and reduces the deflections in the confined connections. This can be attributed to both the confinement of the connection and the increased tightening of the bolts. The connection can also be forced into a yield mode representing a higher load resistance. Splitting at bolted connections is also prevented by the clamps.

(4) The confining clamps allow the damage of the wall to be concentrated in the sheathing-to-framing connections, where it was designed to be taken. By ensuring this failure mode, a more ductile response can be achieved and can more easily be repaired after an earthquake.

(5) The inclusion of an additional reinforcing clamp to connect the end of the sill plate to the wall end post allows the wall to deform with the sill plate, and helps minimize uplift. The additional reinforcing clamp augments the performance of the confining clamps on bolted connections, and improves the performance of the sill plate-to-end post connection.

(6) The repair of a wall outfitted with confining clamps is much easier than repair of an unconfined wall. Since damage to the end post and boundary studs is prevented, those members do not need to be replaced.

5.3 Future Research Studies

More testing should be done on the wood-to-concrete connections, incorporating more specimens to provide a statistically significant sample, and to fully investigate the performance of 2x6 specimens in these connections. The research should focus on those members which are subject to splitting, as was seen with 2x4 members, and the influence of the reinforcing clamp on these members.

Further studies are necessary to examine the performance of plywood shear walls to provide a more statistically significant evaluation. Different configurations and nailing schedules should be tested. A combination clamp which connects the sill plate and the end post, along with the confining clamps on the bolted connections, should be tested for possible inclusion into future construction.

The testing program in this work was performed using quasi-static reversed cyclic loading. Future tests may incorporate some type of random pseudo-dynamic loading similar to that which is experienced in an earthquake. This will show the influence of higher strain rates on the connections and assemblies. Since there are three materials involved in these connections: concrete, wood, and steel, there is a possibility of significant differences in the material's performance under a high strain rate. If the load is

applied as a sudden impact, it is possible splitting would be even more likely to occur. This would strengthen the need for the confining clamps.

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APPENDIX A

NOMENCLATURE

The following symbols are used in this paper:

a	=	height of shear wall
b	=	length of shear wall
C_D	=	load duration factor
C_G	=	group action factor for connections
C_M	=	wet service factor
C_t	=	temperature factor
C_Δ	=	geometry factor for connections
d	=	deflection of a shear wall
G'	=	shear stiffness of a shear wall, kips/inch
o/c	=	on-center, or center-to-center
P	=	resultant lateral load at top of shear wall
S.G.	=	specific gravity of wood species
t_m	=	thickness of main member, inches
t_s	=	thickness of side member, inches
$Z_{\parallel}$	=	nominal lateral design value for a single bolt or lag screw connection with all wood members loaded parallel to grain, lbs
ϕ	=	diameter of bolt, inches

APPENDIX B

FIGURES OF EXPERIMENTAL RESPONSE FOR

TASK 1: SILL PLATE COMPONENT TESTS

The following figures represent the force-deformation and energy absorption response for all wood-to-concrete connection specimens.

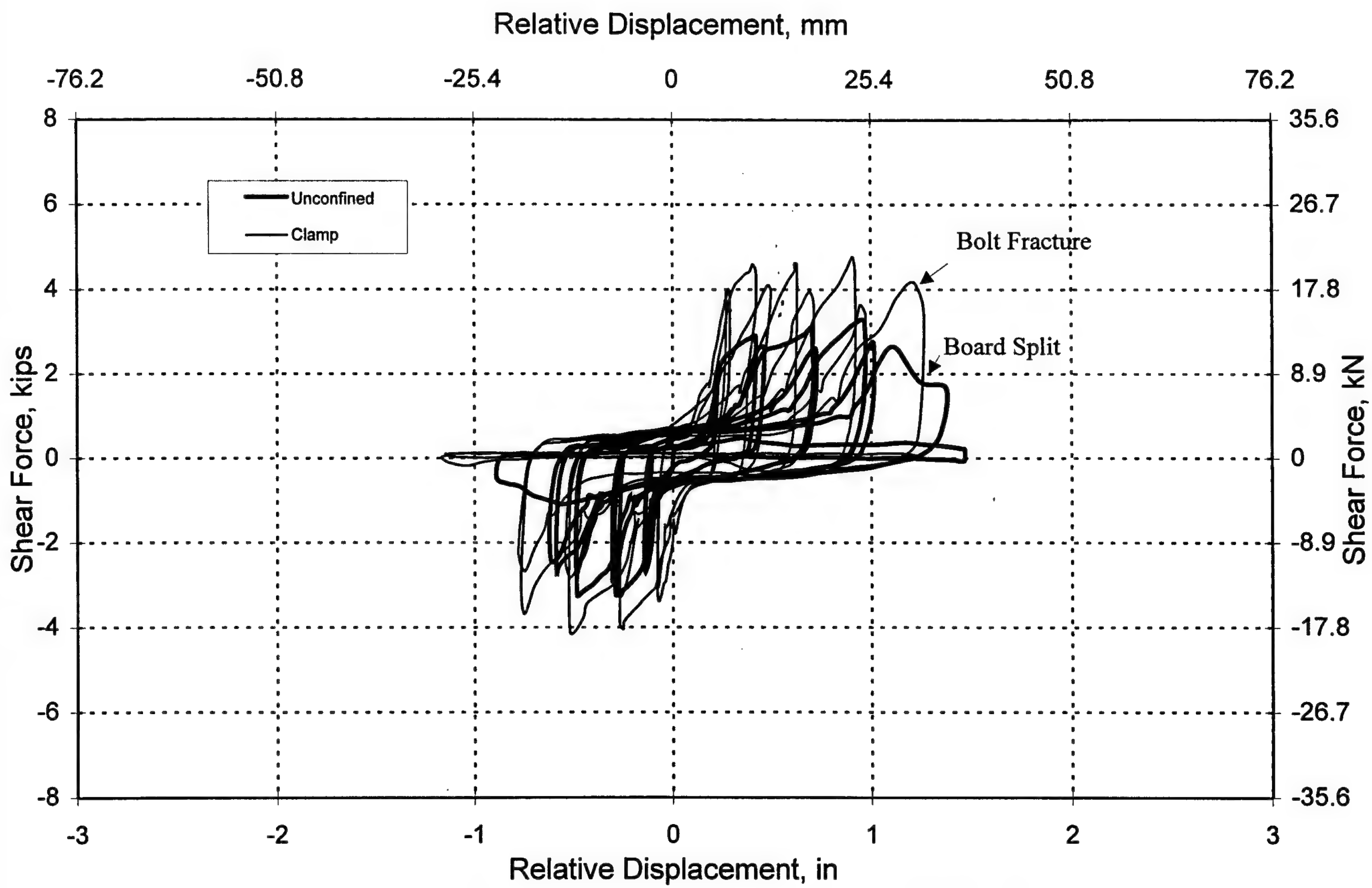

FIG. B1. Load-Displacement Response: 2x4 with 1/2" Diameter Bolt

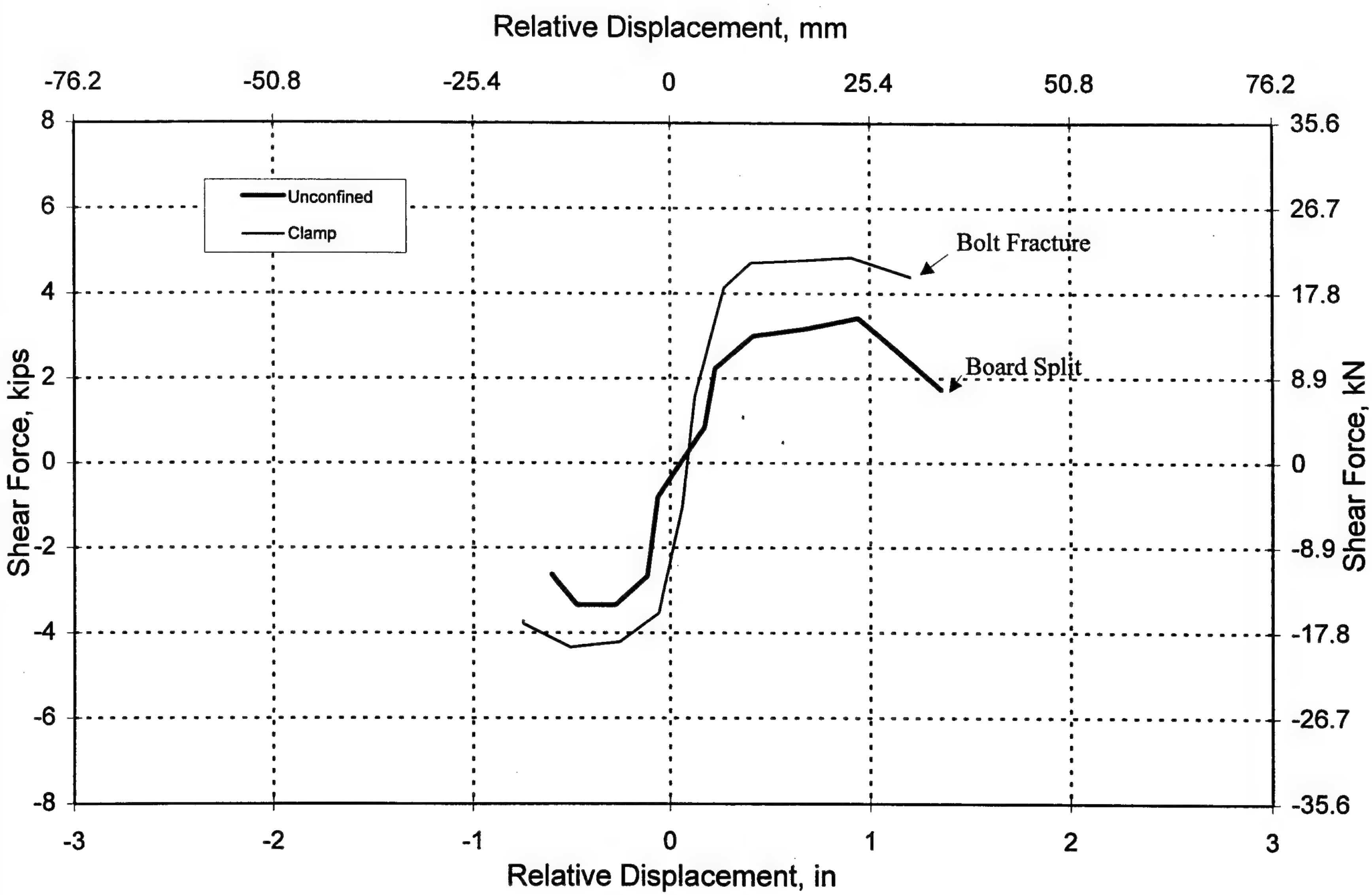

FIG. B2. Load-Displacement Response Envelope: 2x4 with 1/2" Diameter Bolt

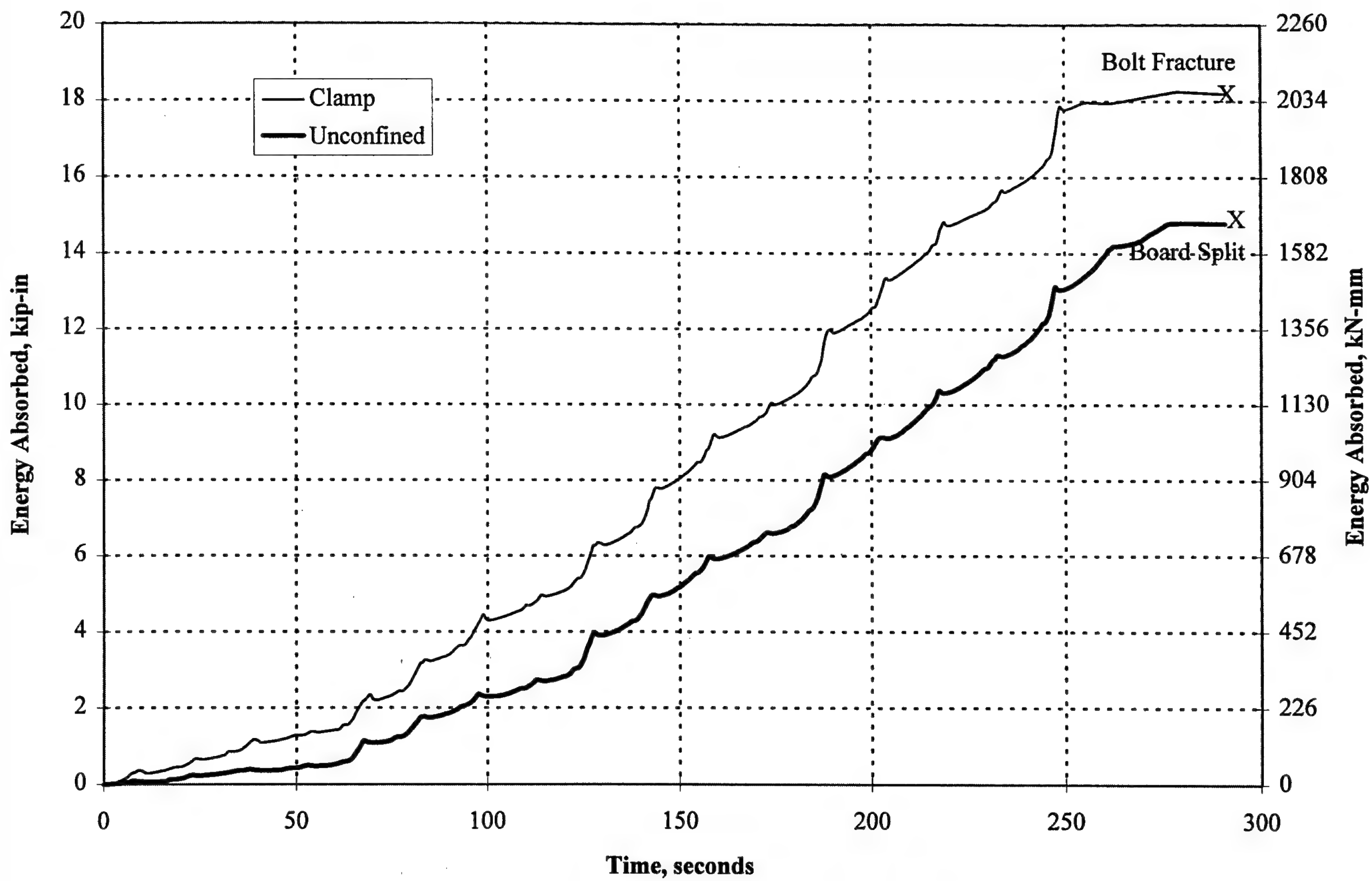

FIG. B3. Energy Dissipation: 2x4 with 1/2" Diameter Bolt

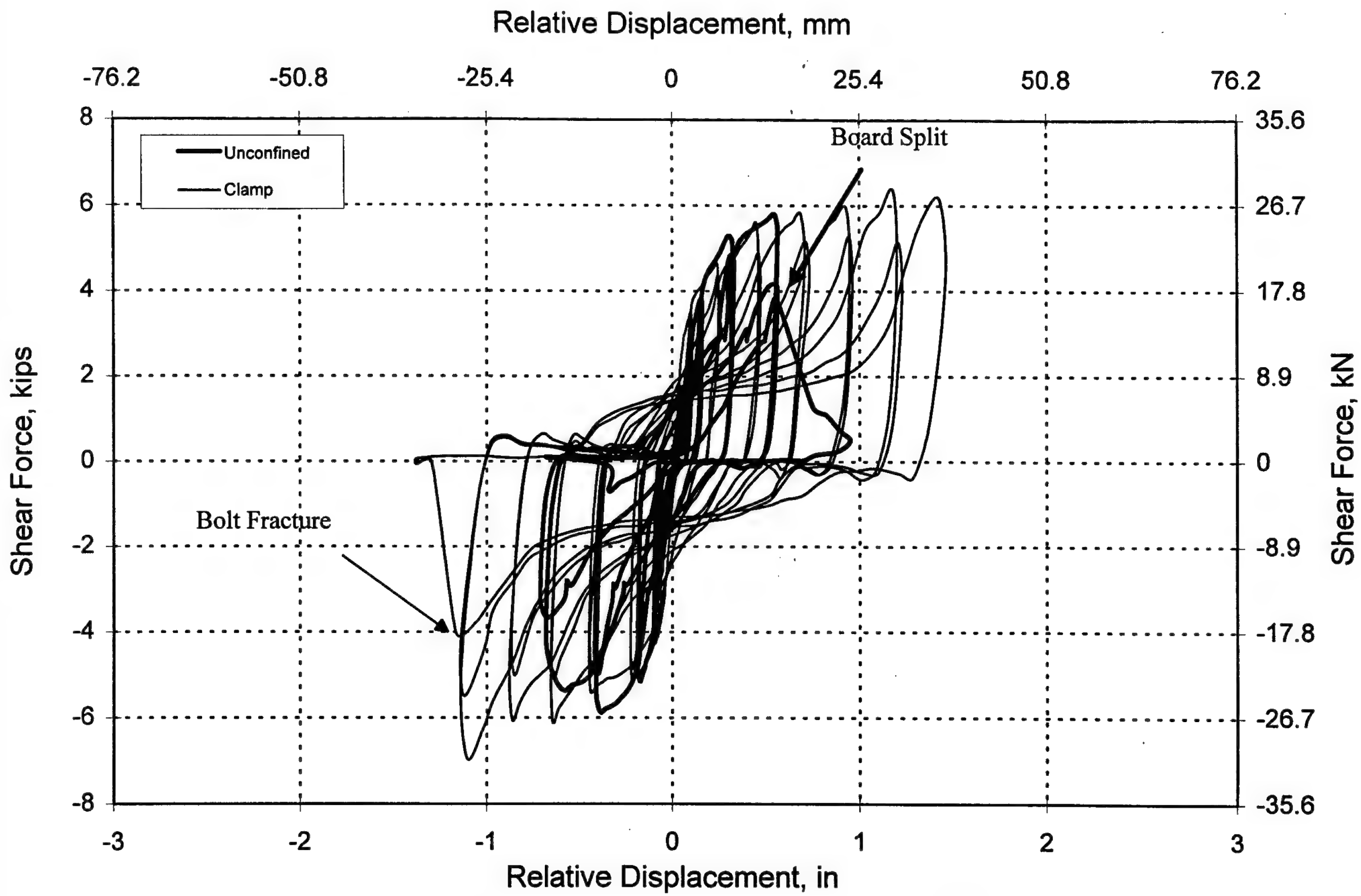

FIG. B4. Load-Displacement Response: 2x4 with 3/4" Diameter Bolt

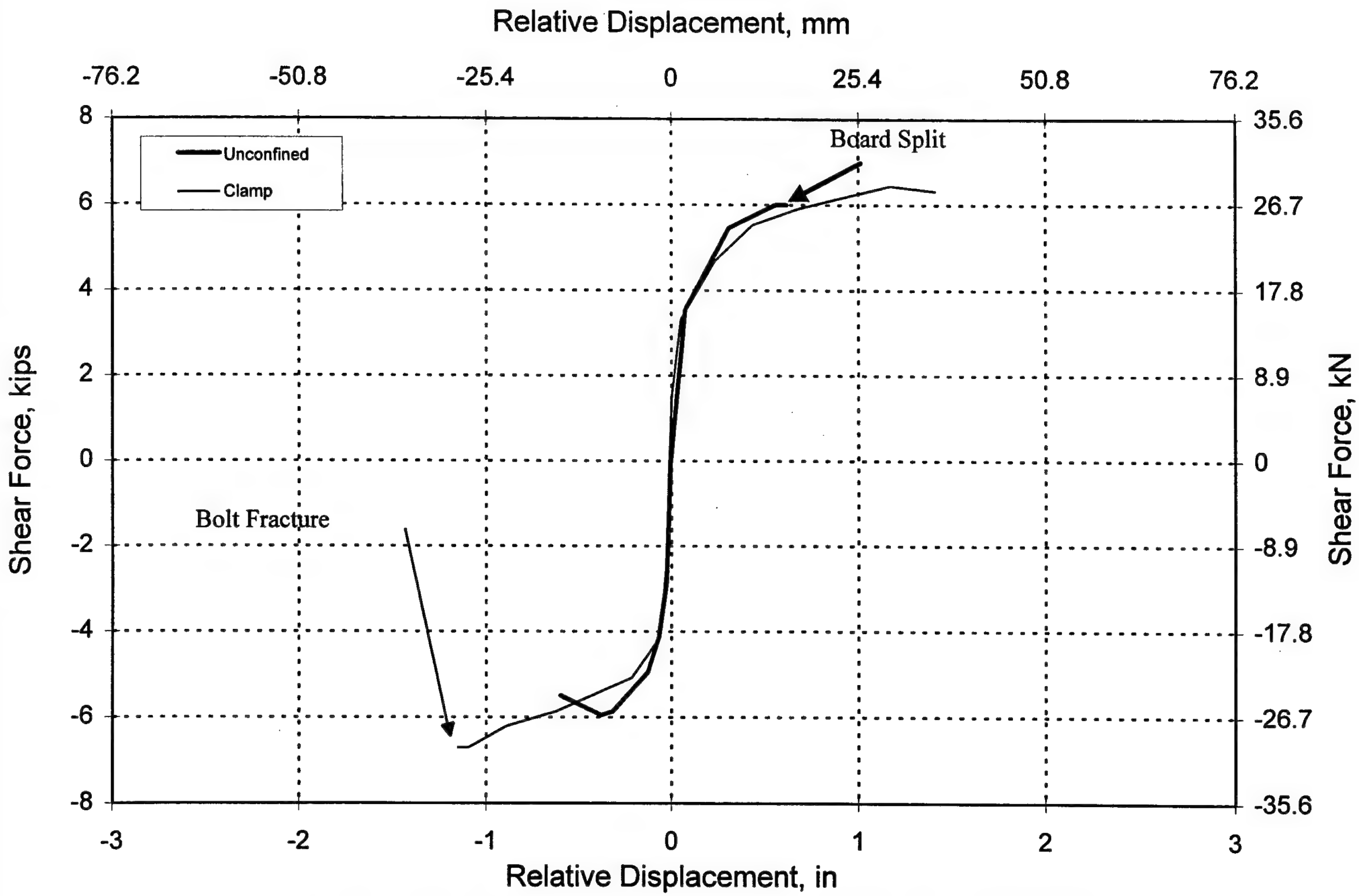

FIG. B5. Load-Displacement Response Envelope: 2x4 with 3/4" Diameter Bolt

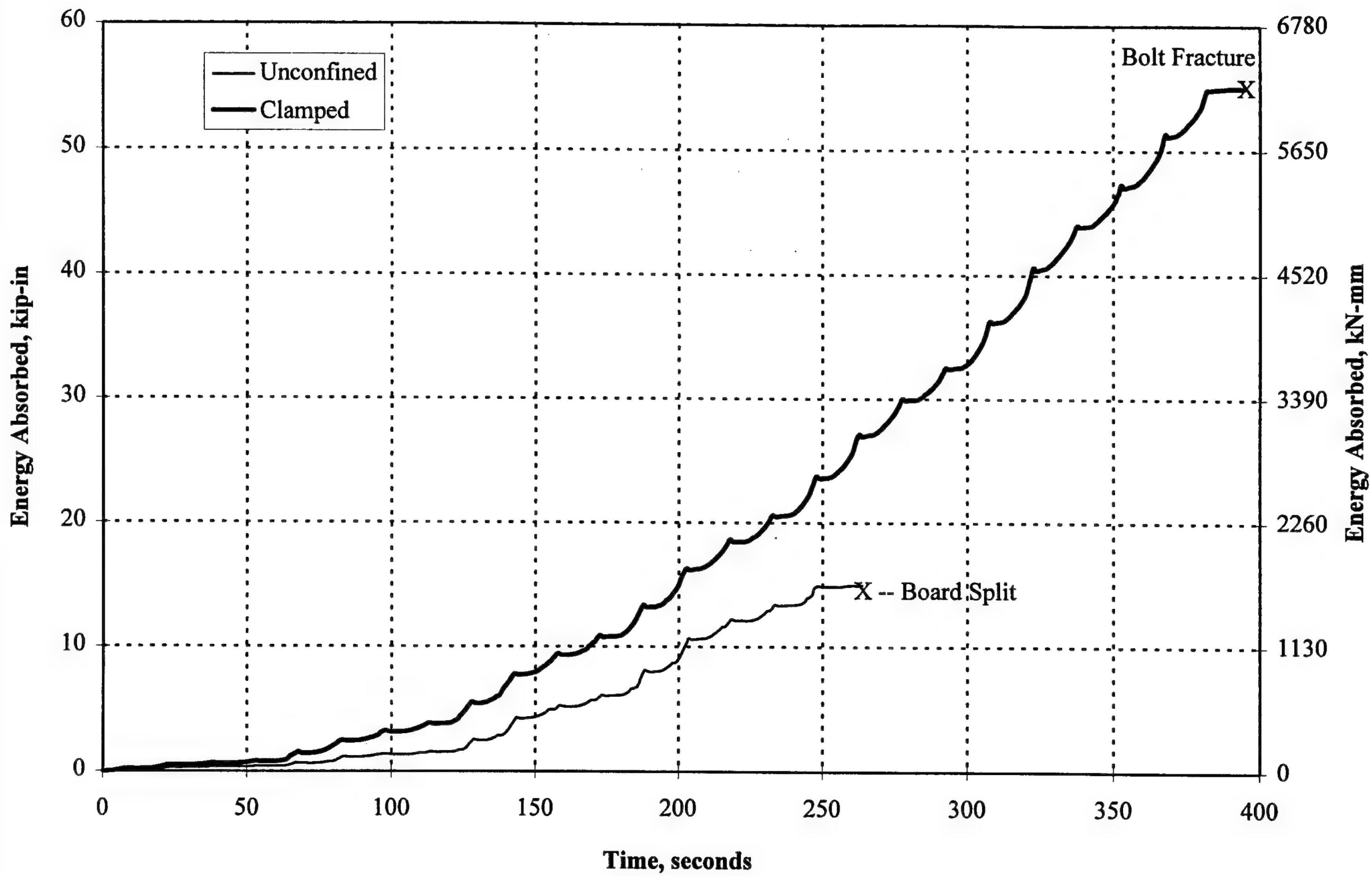

FIG. B6. Energy Dissipation: 2x4 with 3/4" Diameter Bolt

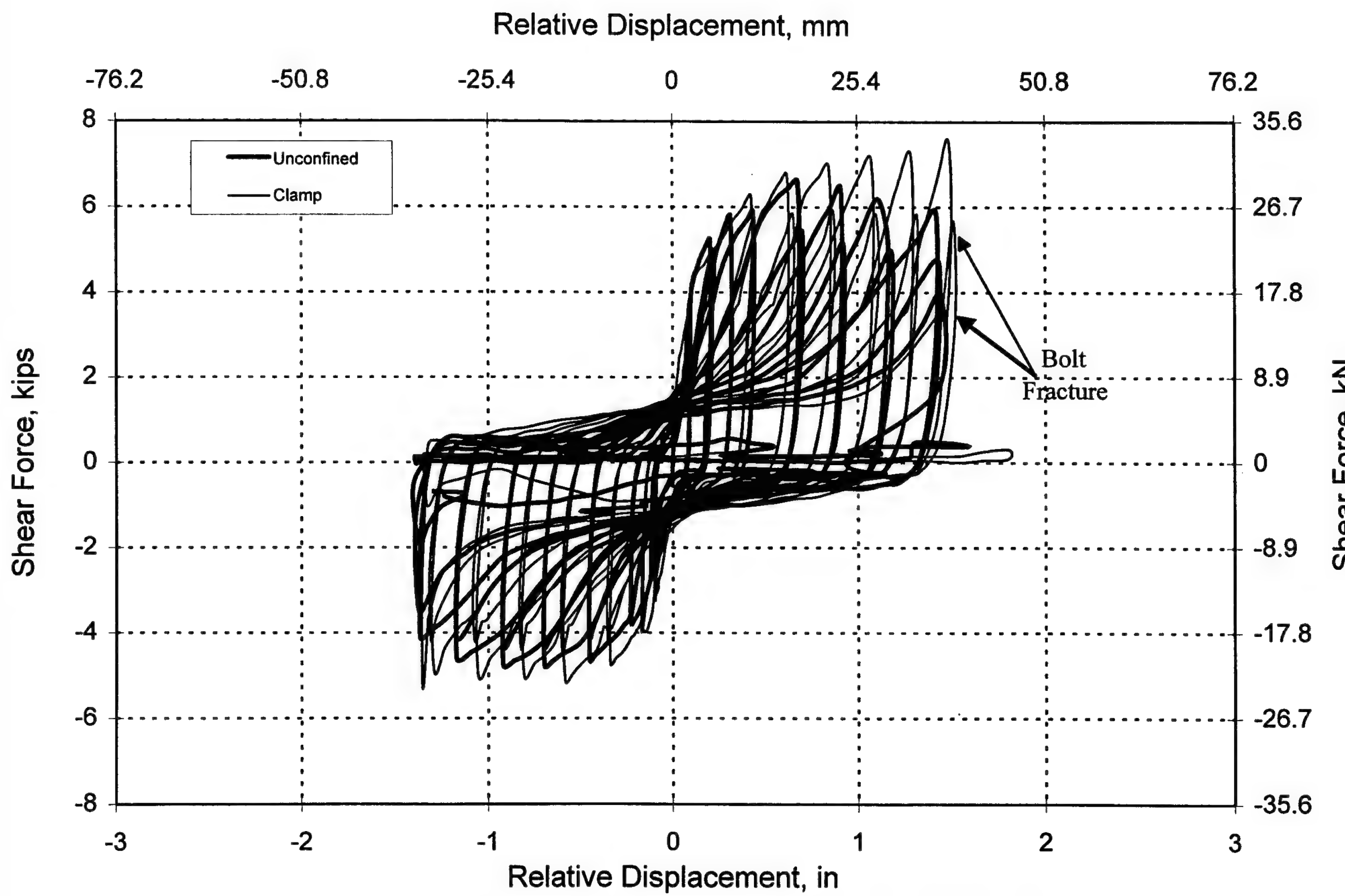

FIG. B7. Load-Displacement Response: 3x4 with 3/4" Diameter Bolt

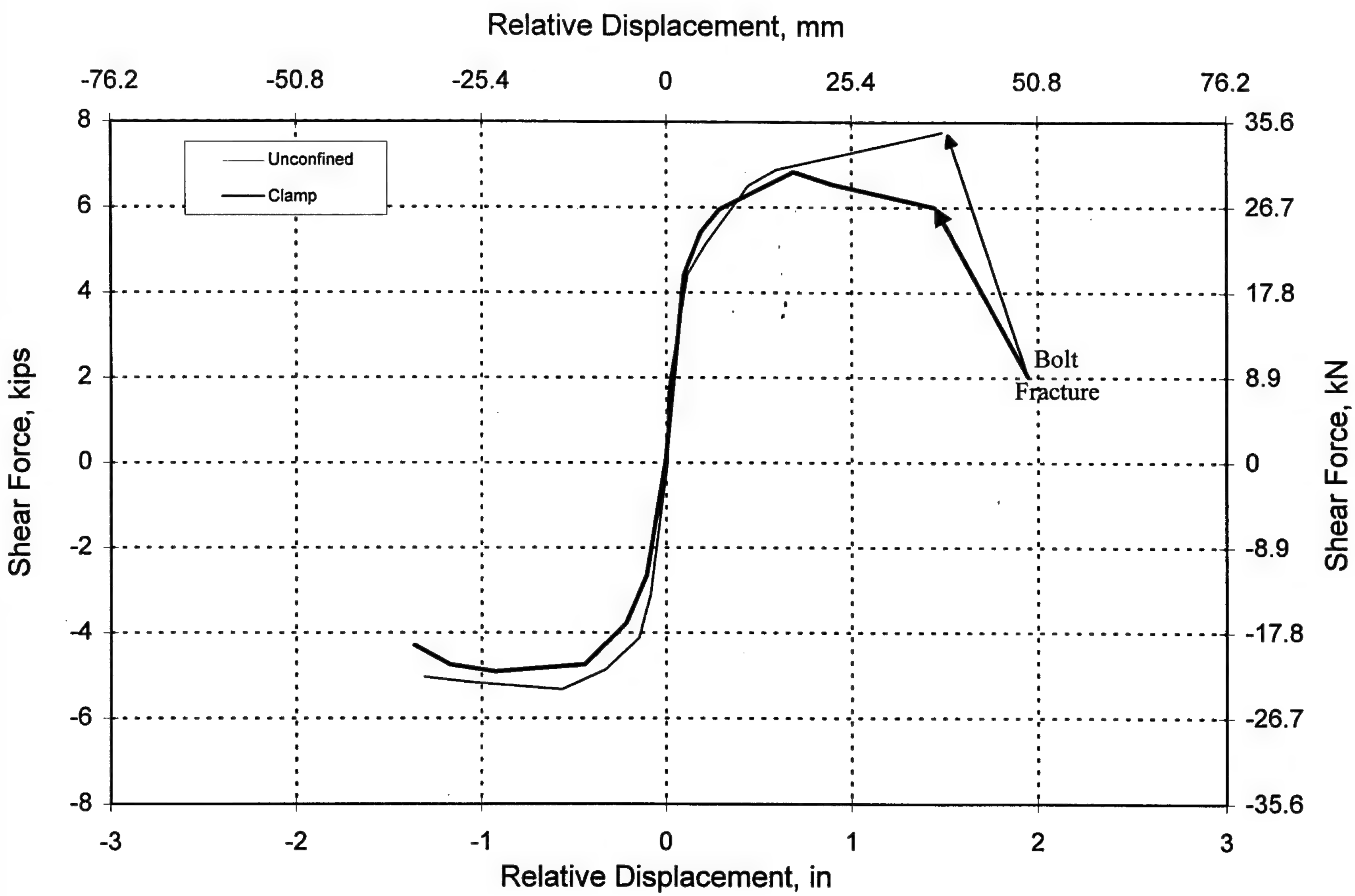

FIG. B8. Load-Displacement Response Envelope: 3x4 with 3/4" Diameter Bolt

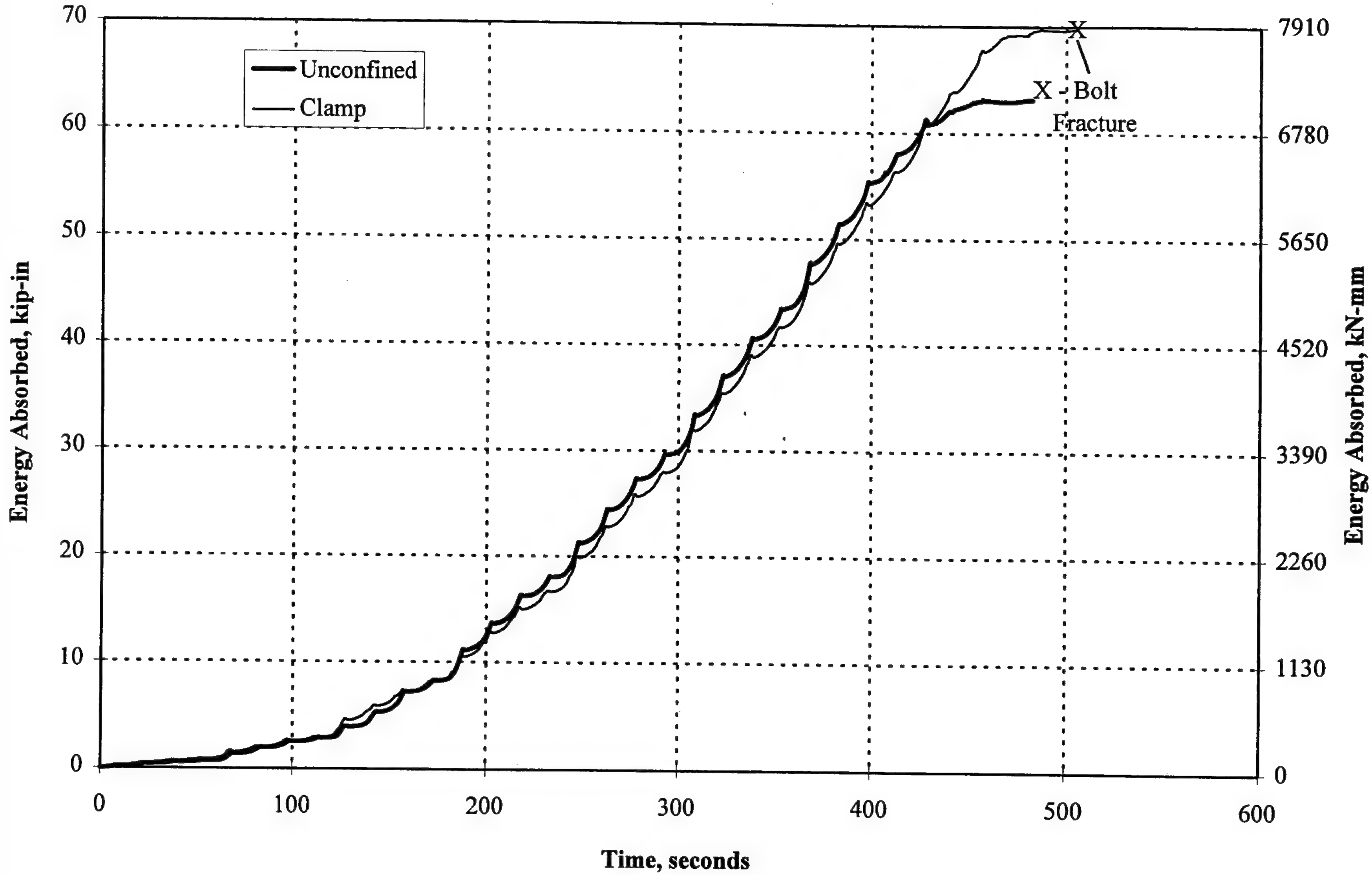

FIG. B9. Energy Dissipation: 3x4 with 3/4" Diameter Bolt

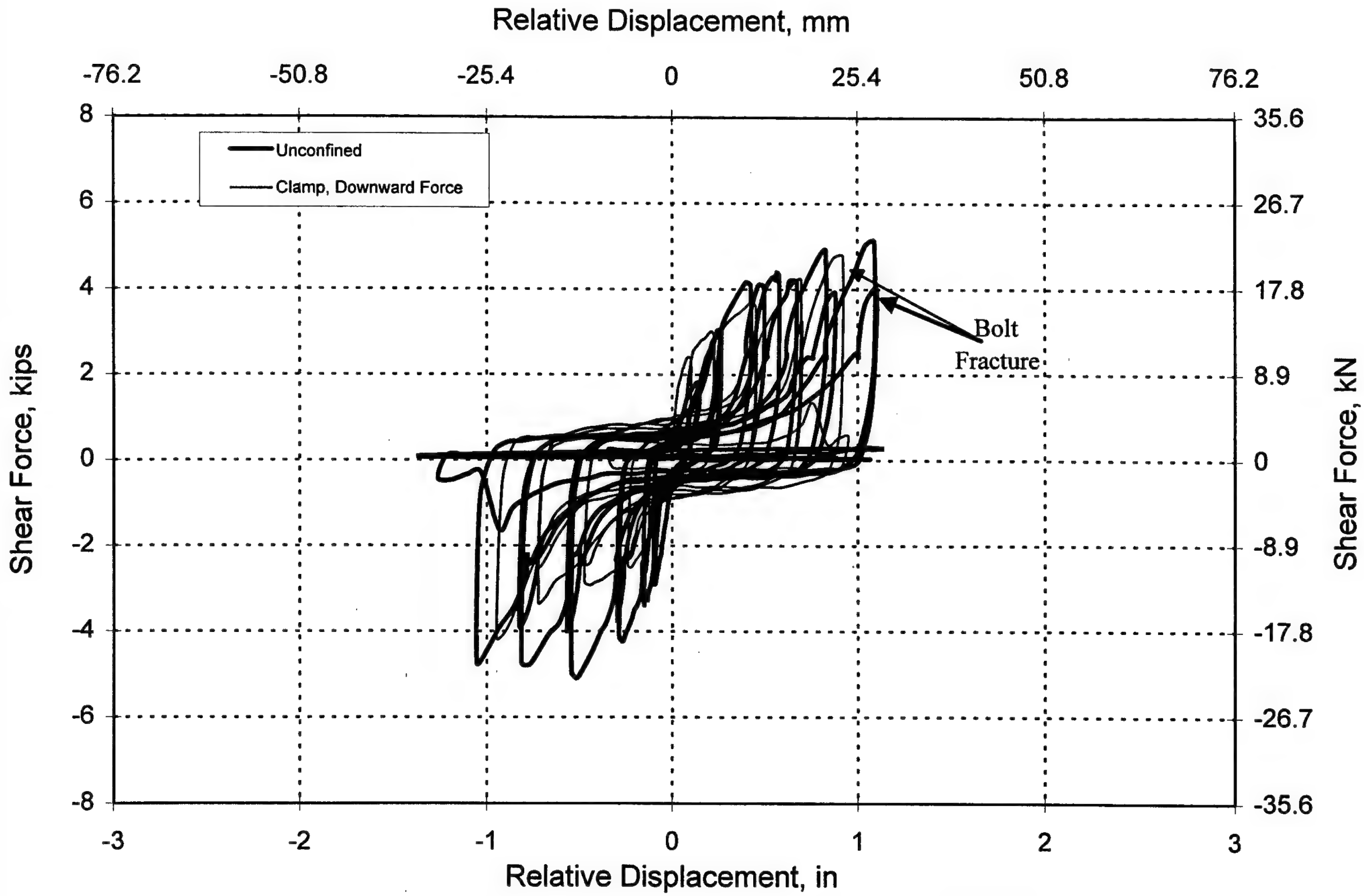

FIG. B10. Load-Displacement Response: 2x6 with 1/2" Diameter Bolt

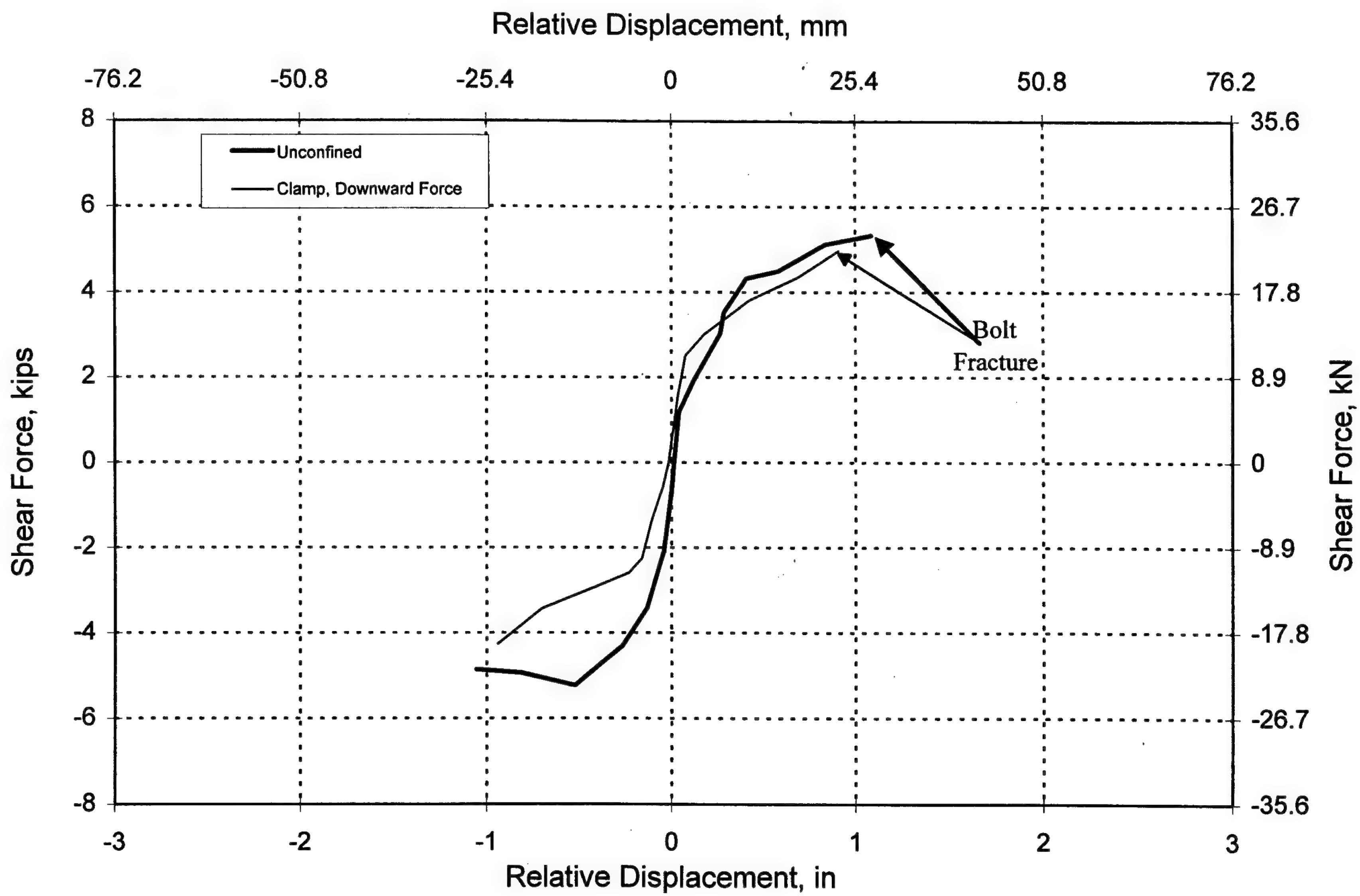

FIG. B11. Load-Displacement Response Envelope: 2x6 with 1/2" Diameter Bolt

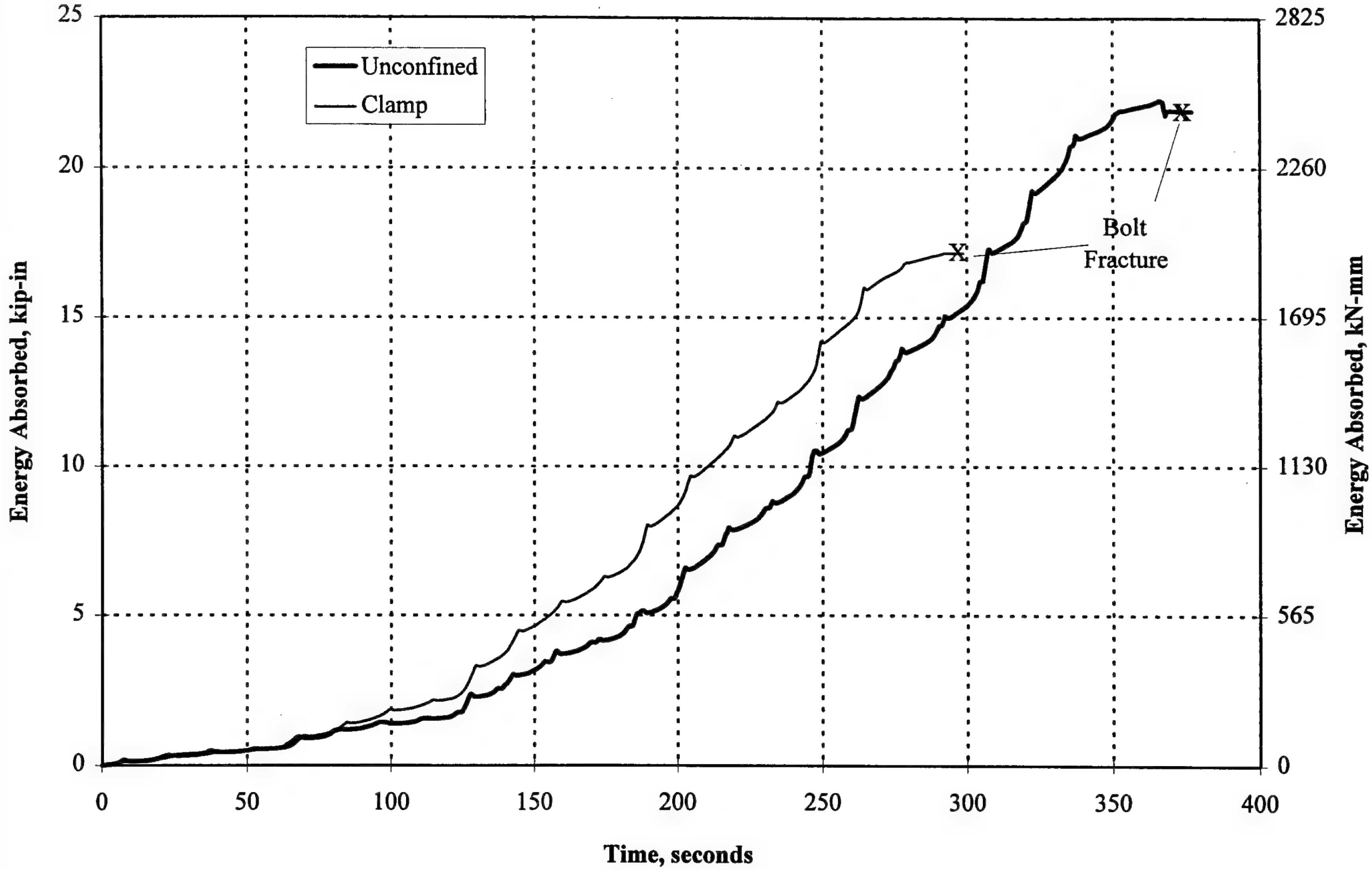

FIG. B12. Energy Dissipation: 2x6 with 1/2" Diameter Bolt

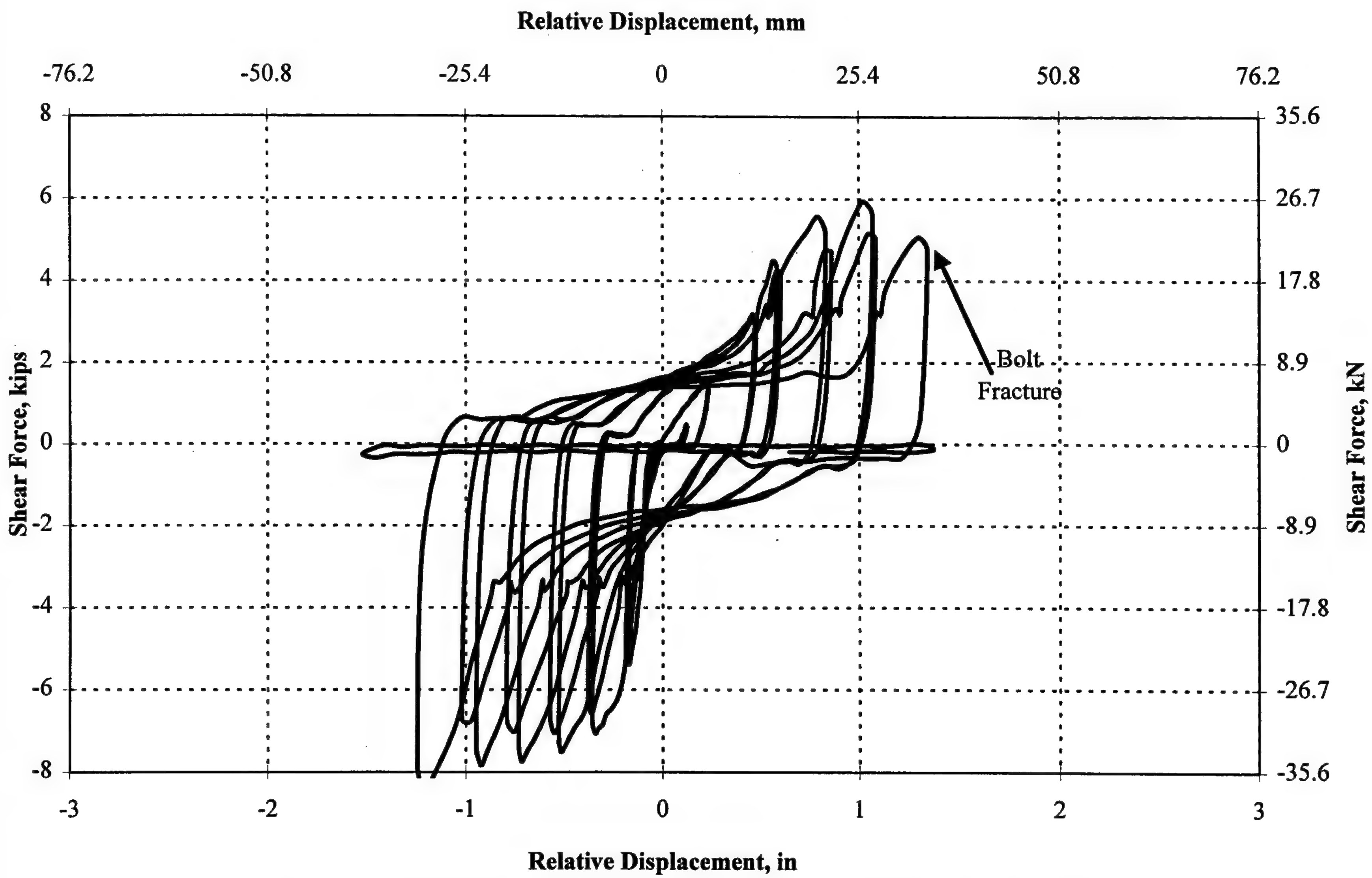

FIG. B13. Load-Displacement Response: 2x6 with 3/4" Diameter Bolt (Unconfined)

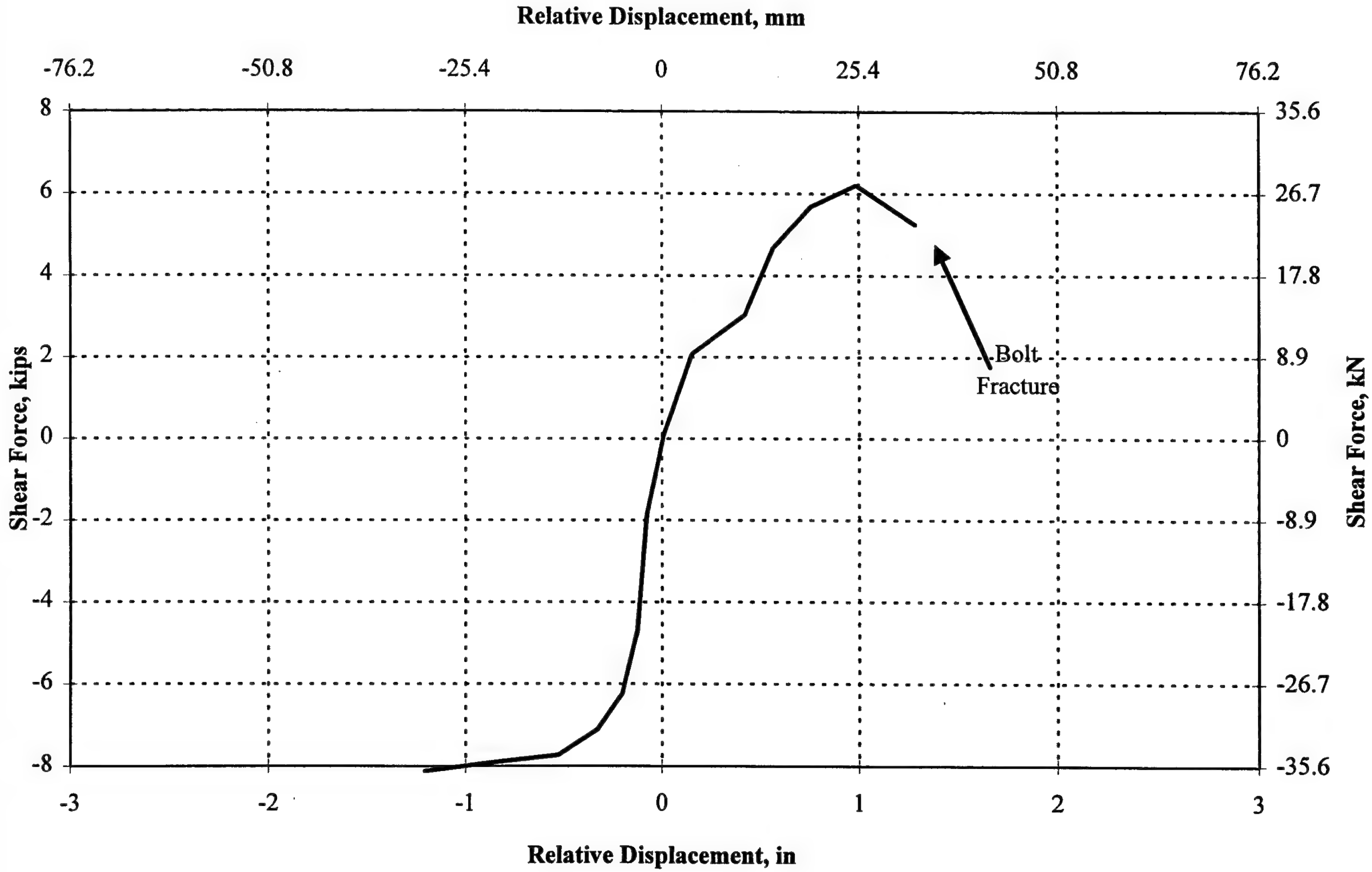

FIG. B14. Load-Displacement Response Envelope: 2x6 with 3/4" Diameter Bolt (Unconfined)

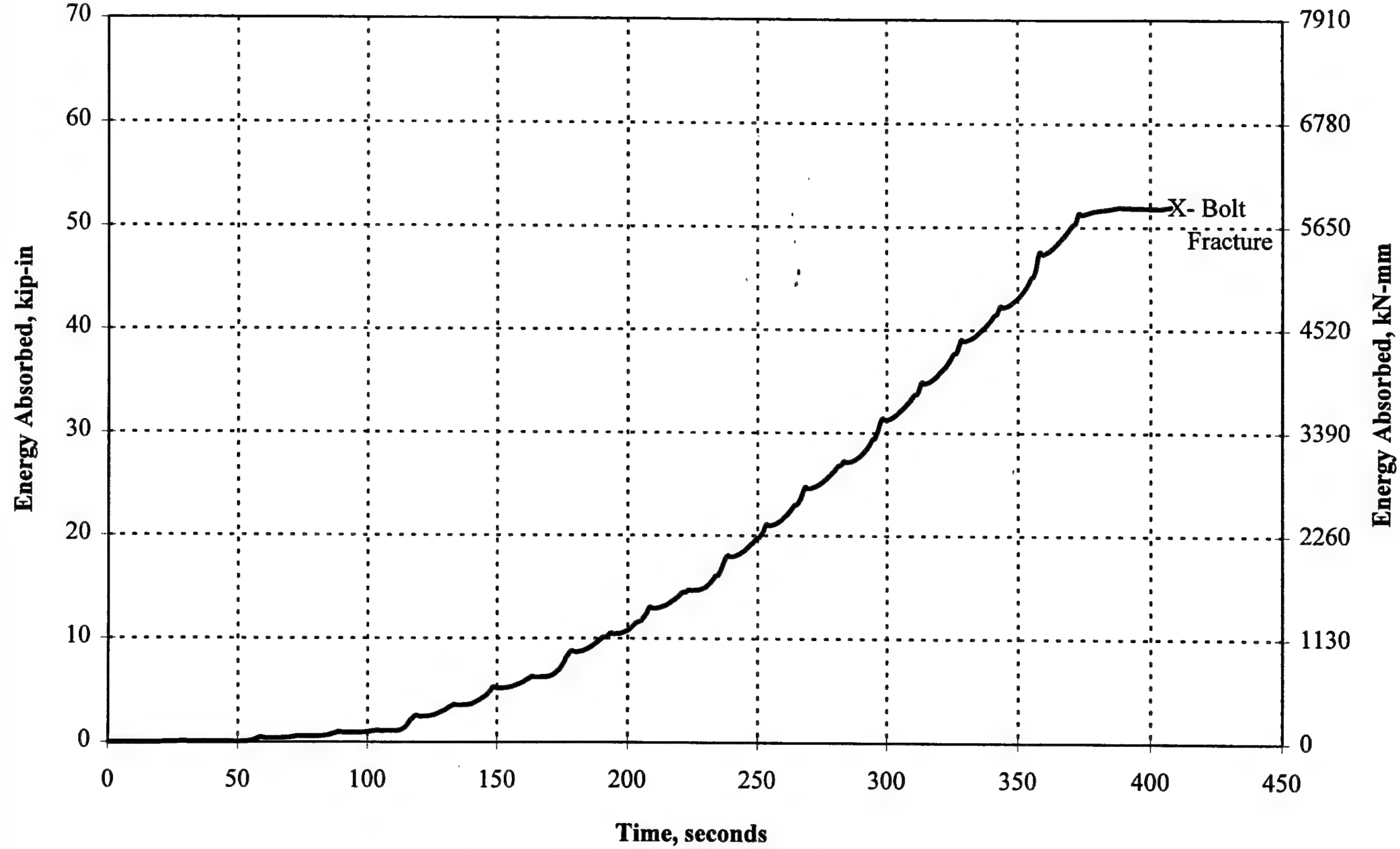

FIG. B15. Energy Absorption: 2x6 with 3/4" Diameter Bolt (Unconfined)

APPENDIX C

FIGURES OF EXPERIMENTAL RESPONSE FOR

TASK 2: FULL SCALE PLYWOOD SHEAR WALL TESTS

The following figures represent the force-deformation response for all full scale plywood shear wall tests.

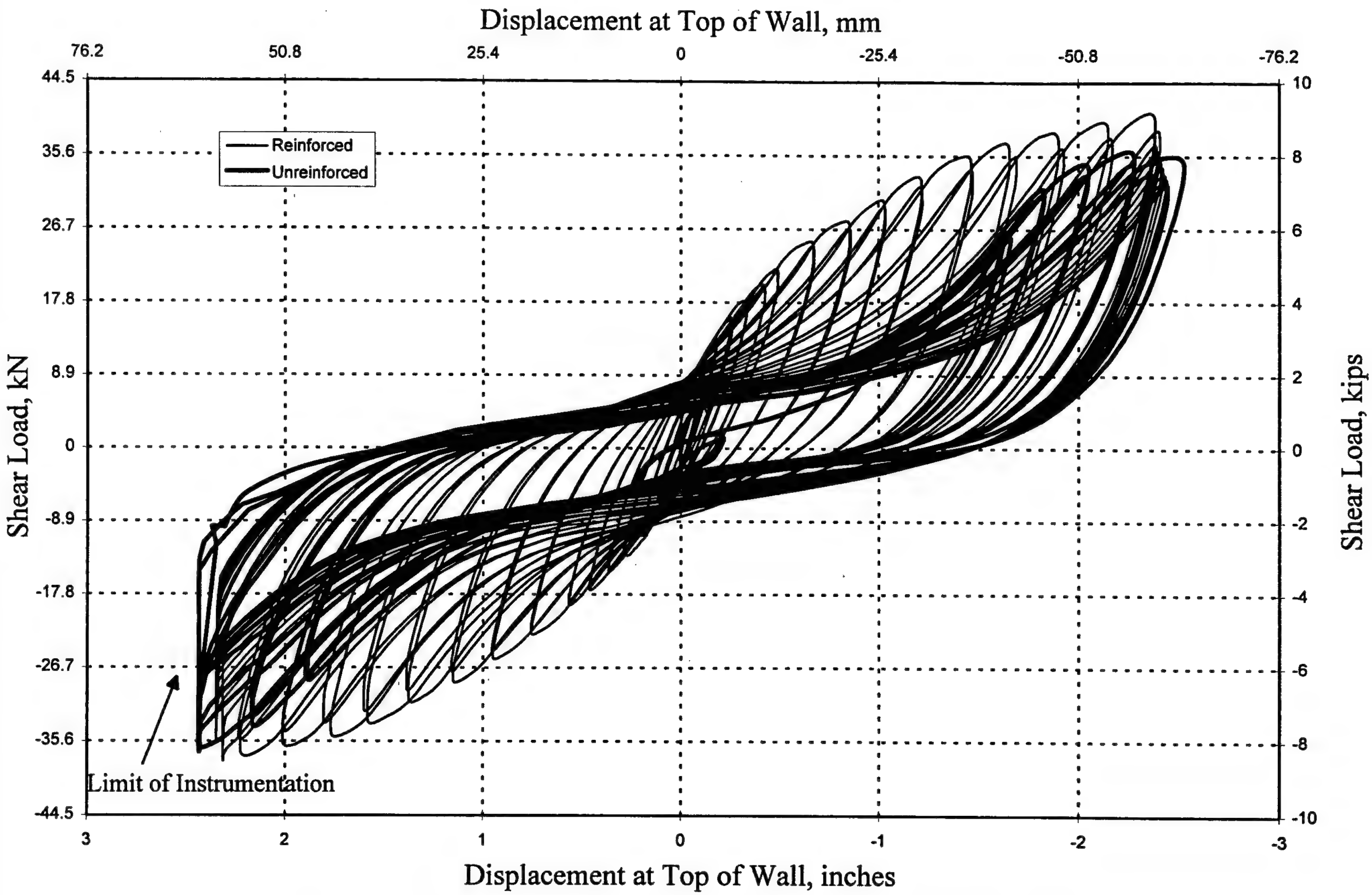

FIG. C1. Load-Displacement Response: Unreinforced and Reinforced Walls

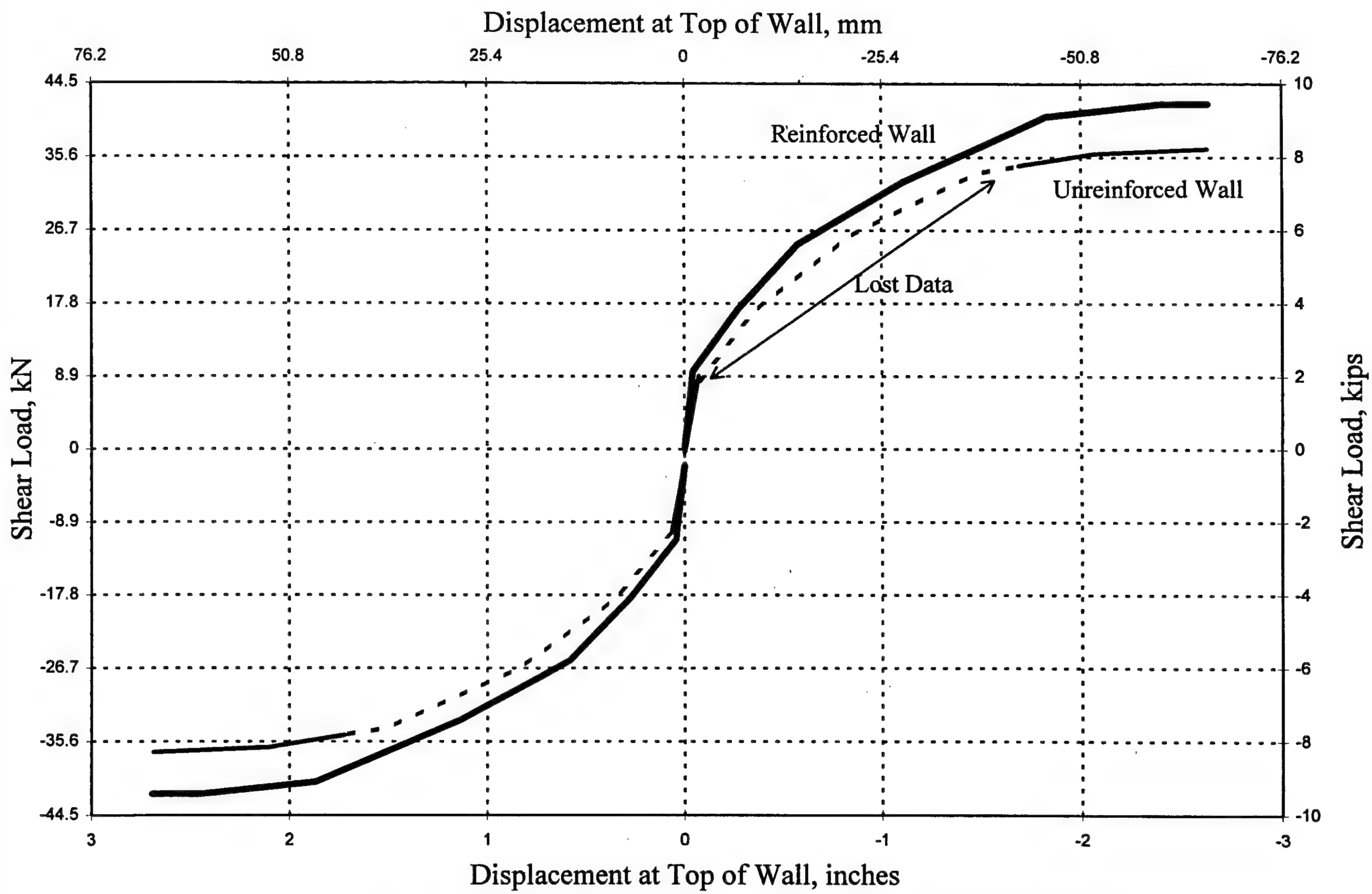

FIG. C2. Load-Displacement Response Envelopes: Unreinforced and Reinforced Walls

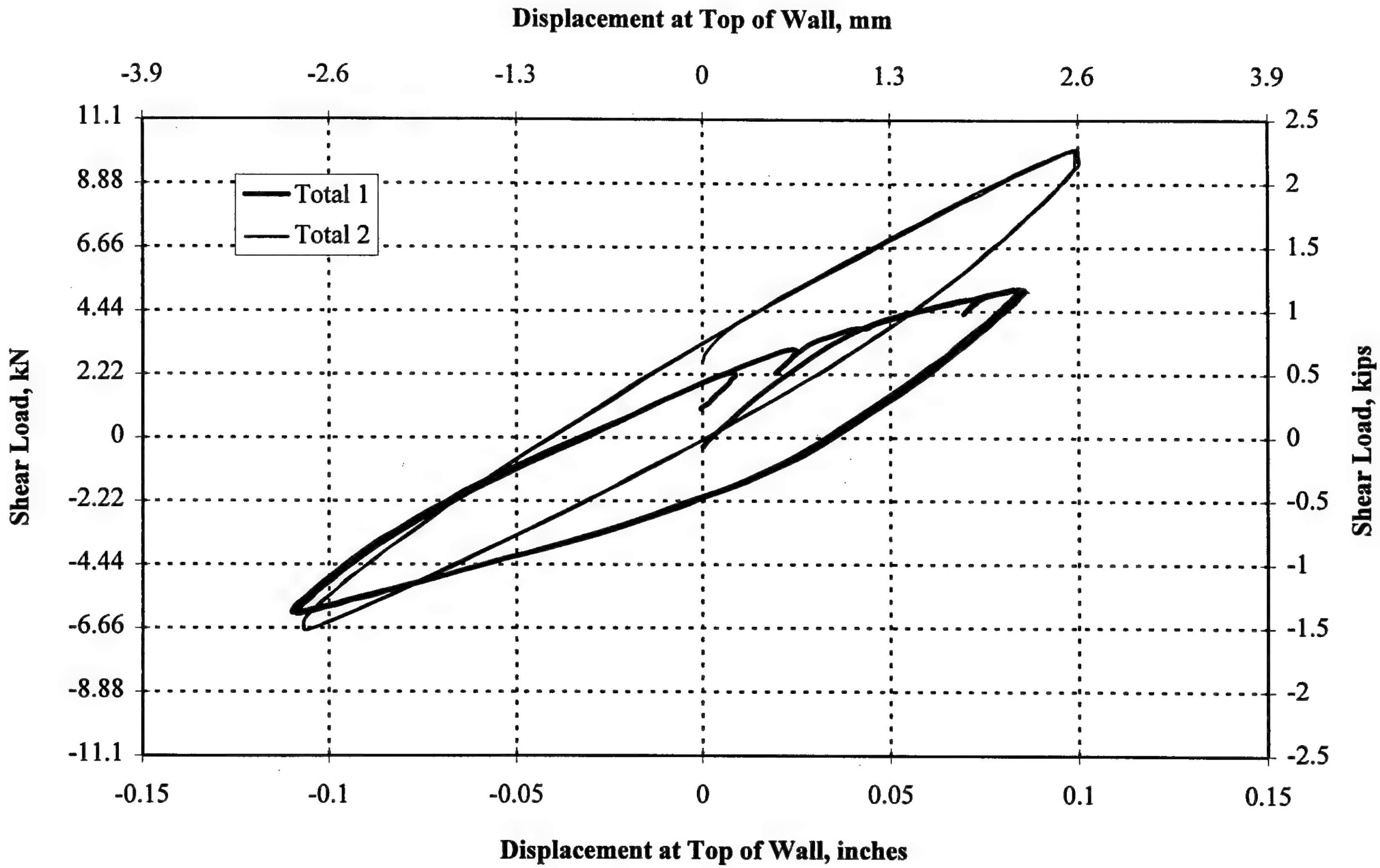

FIG. C3. Total Load-Displacement Response, Initial Deflection
(1 = Unreinforced, 2 = Reinforced)

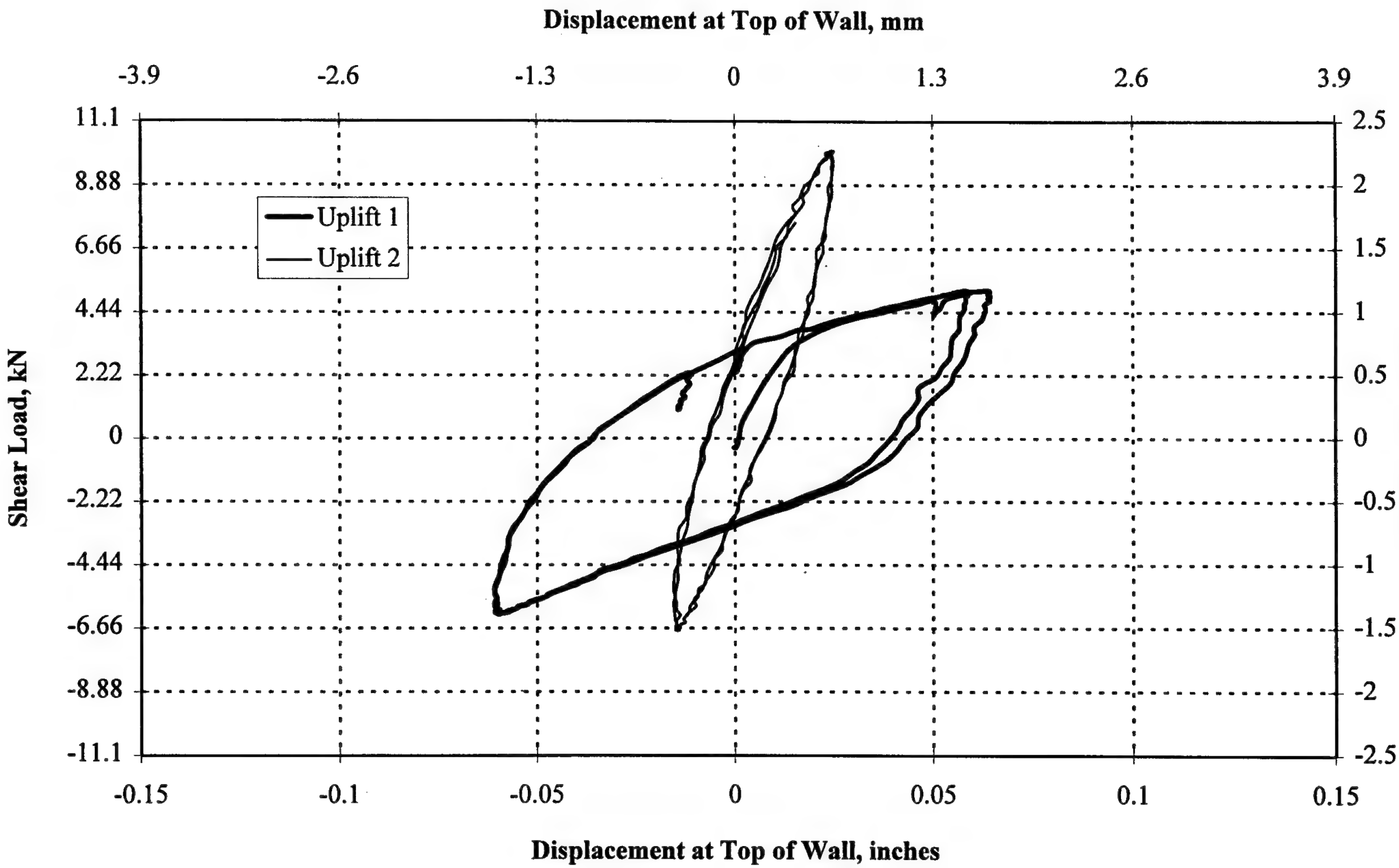

FIG. C4. Overturning Load-Displacement Response, Initial Deflection
(1 = Unreinforced, 2 = Reinforced)

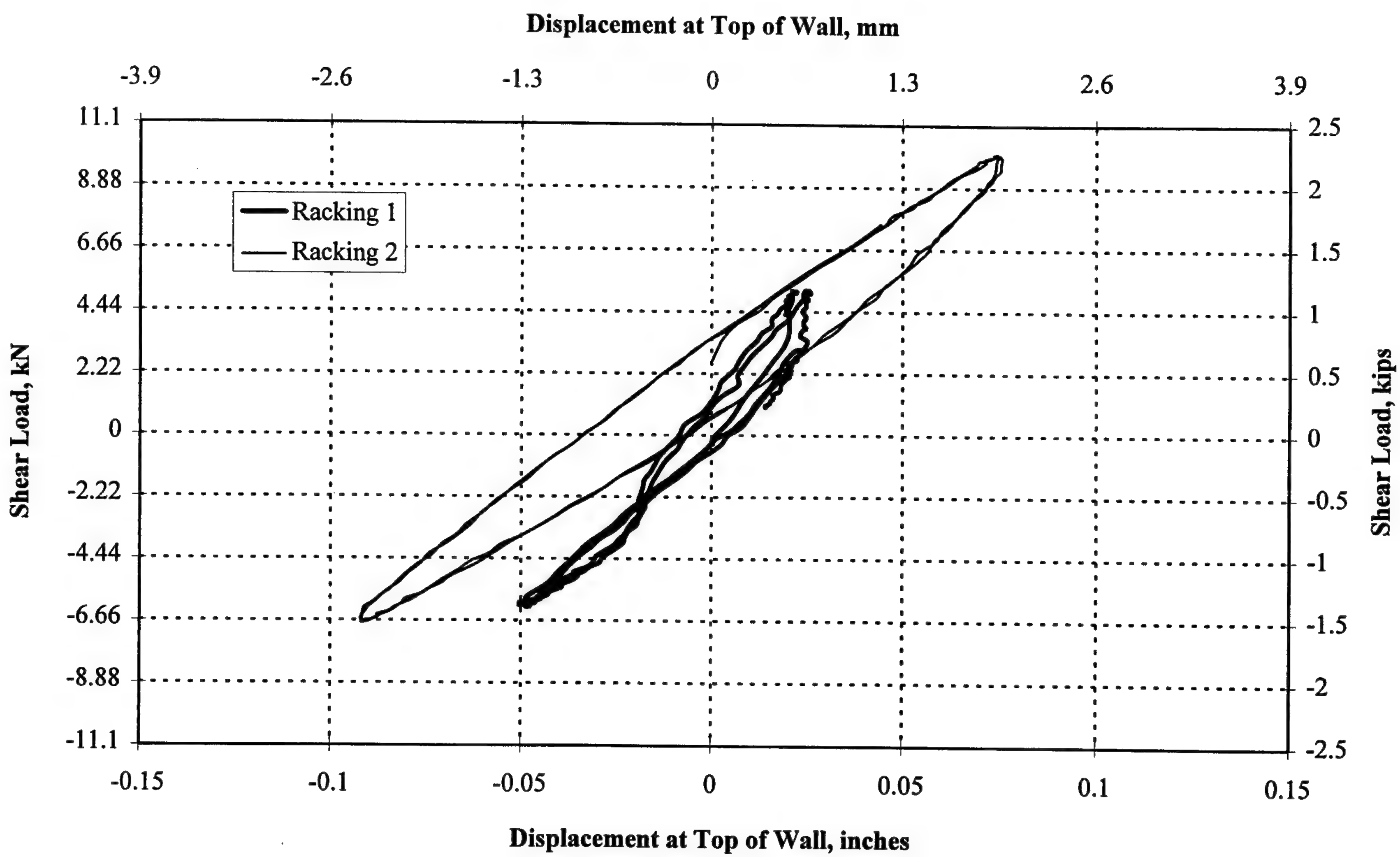

FIG. C5. Shear/Bending Load-Displacement Response, Initial Deflection
(1 = Unreinforced, 2 = Reinforced)

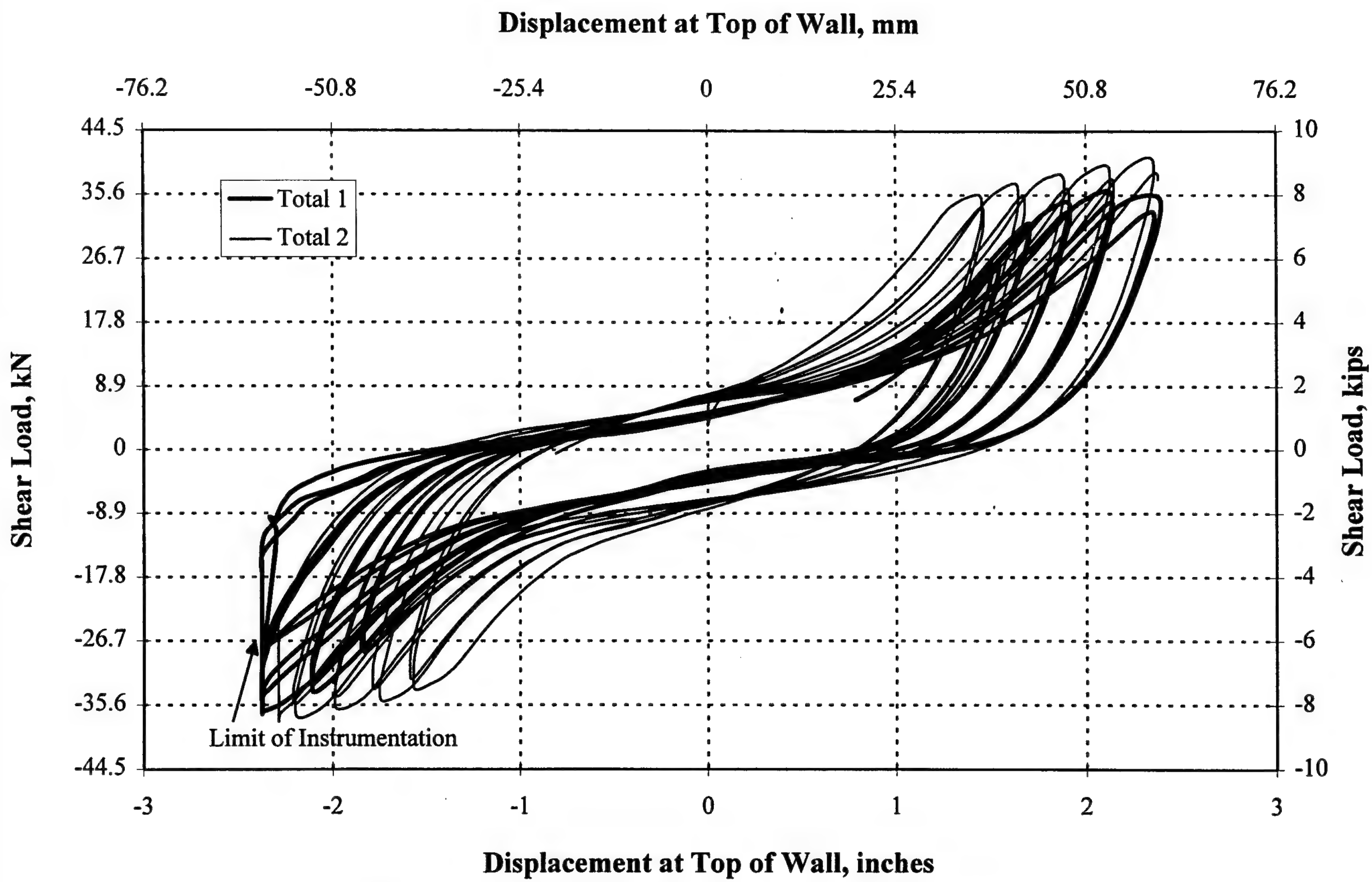

FIG. C6. Total Load-Displacement Response, Large Deformation
(1 = Unreinforced, 2 - Reinforced)

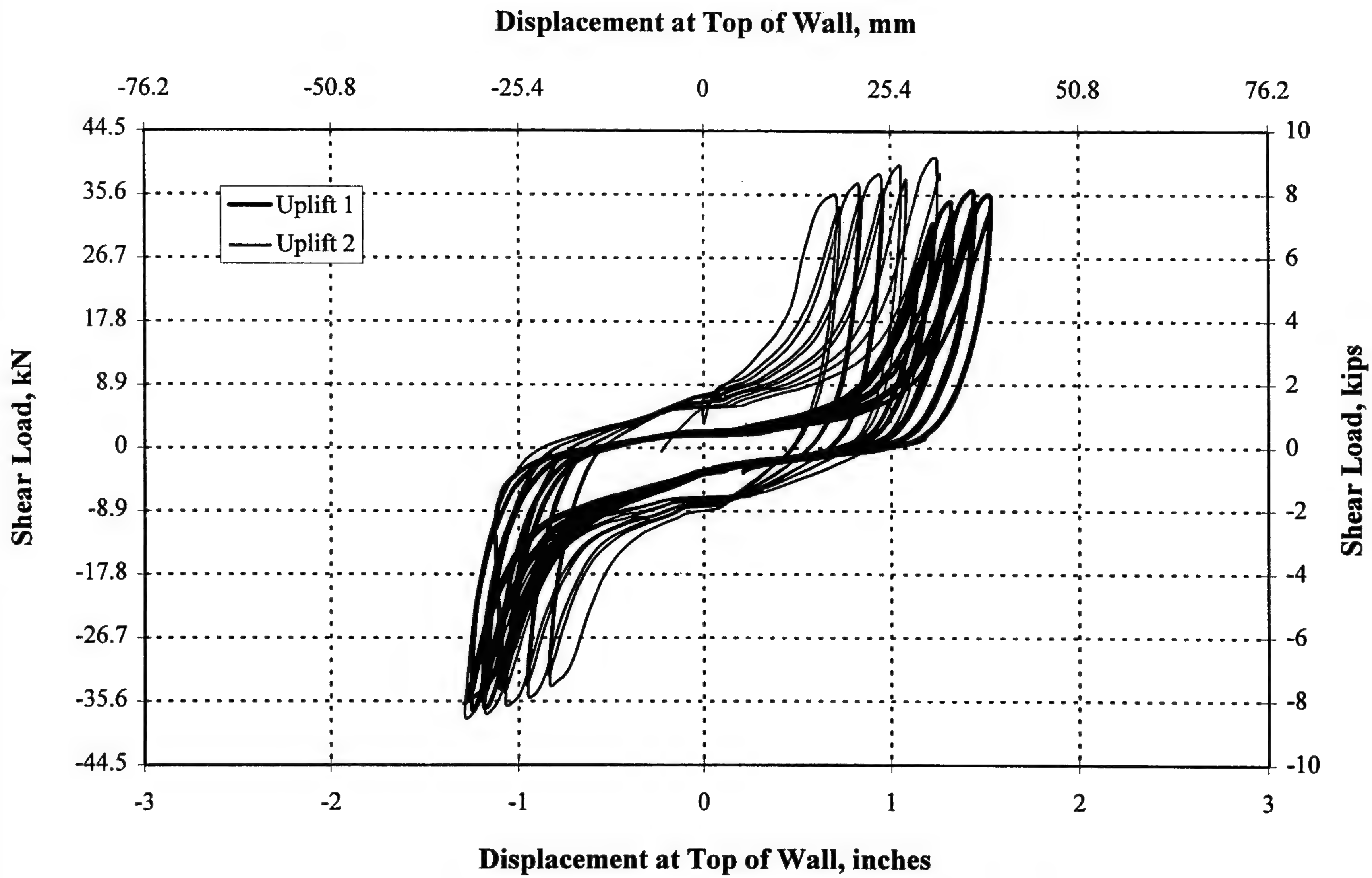

FIG. C7. Overturning Load-Displacement Response, Large Deformation
(1 = Unreinforced, 2 = Reinforced)

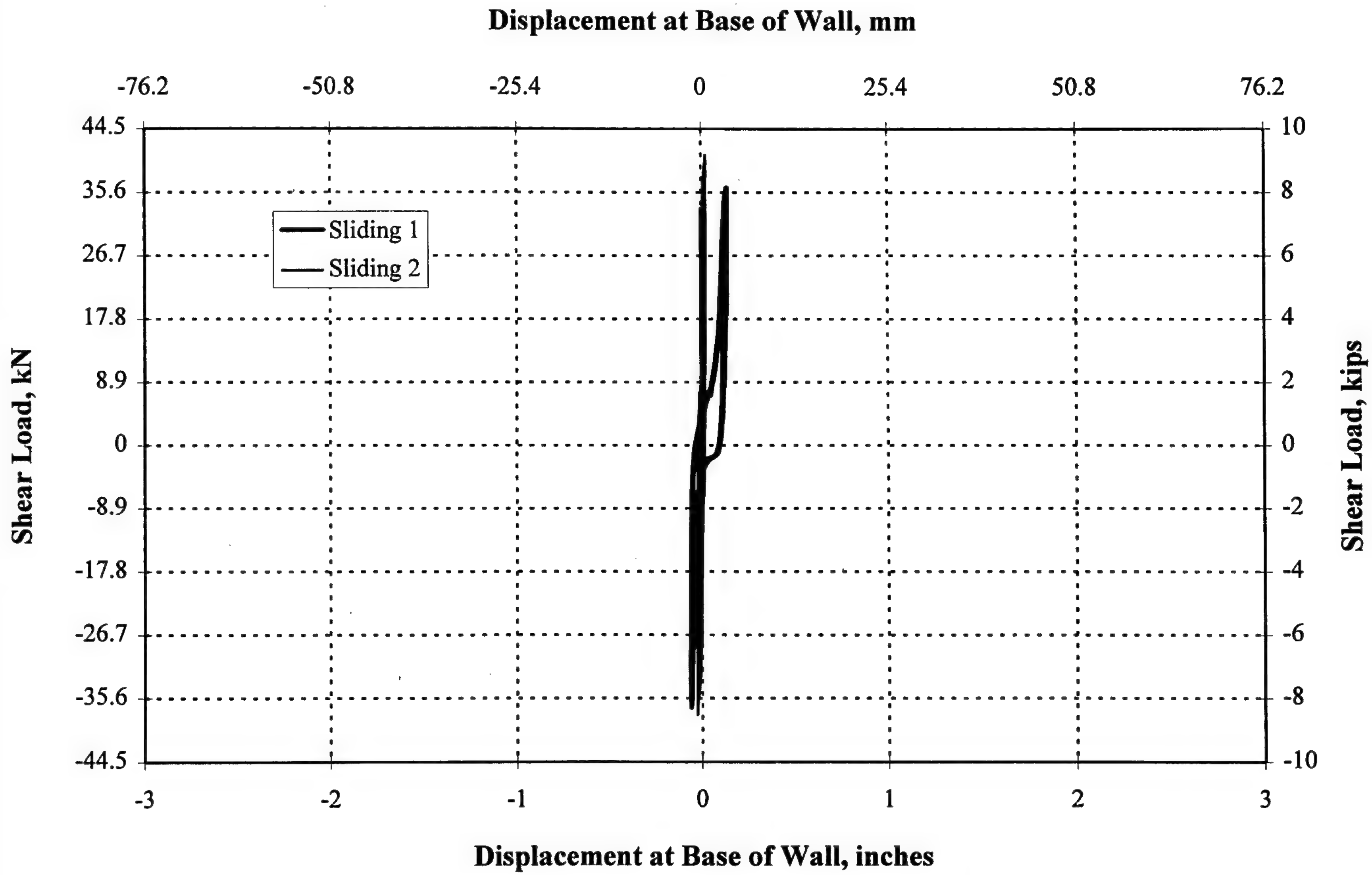

FIG. C8. Sliding Load-Displacement Response, Large Deformation
(1 = Unreinforced, 2 = Reinforced)

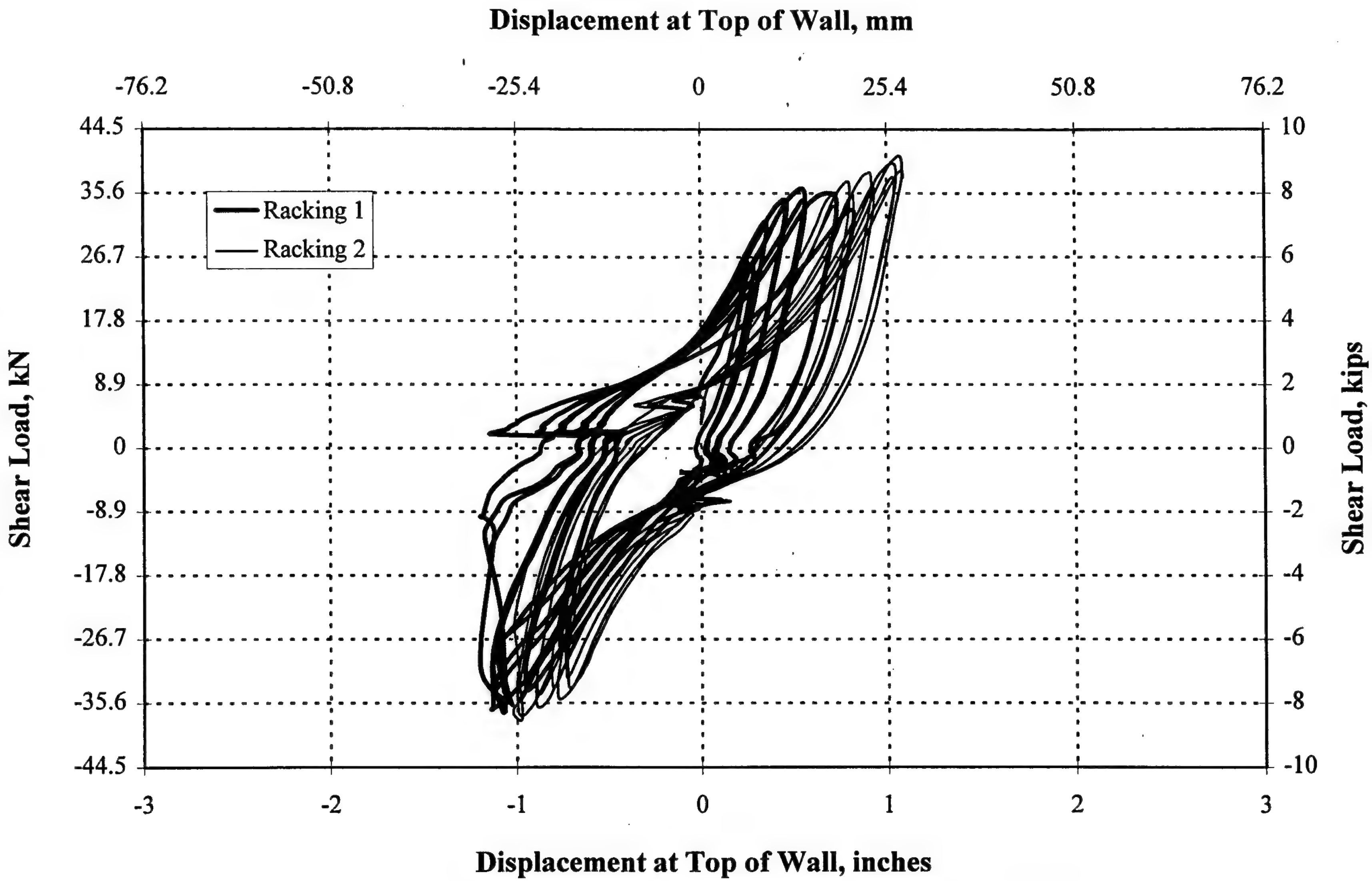

FIG. C9. Shear/Bending Load-Displacement Response, Large Deformation
(1 = Unreinforced, 2 = Reinforced)

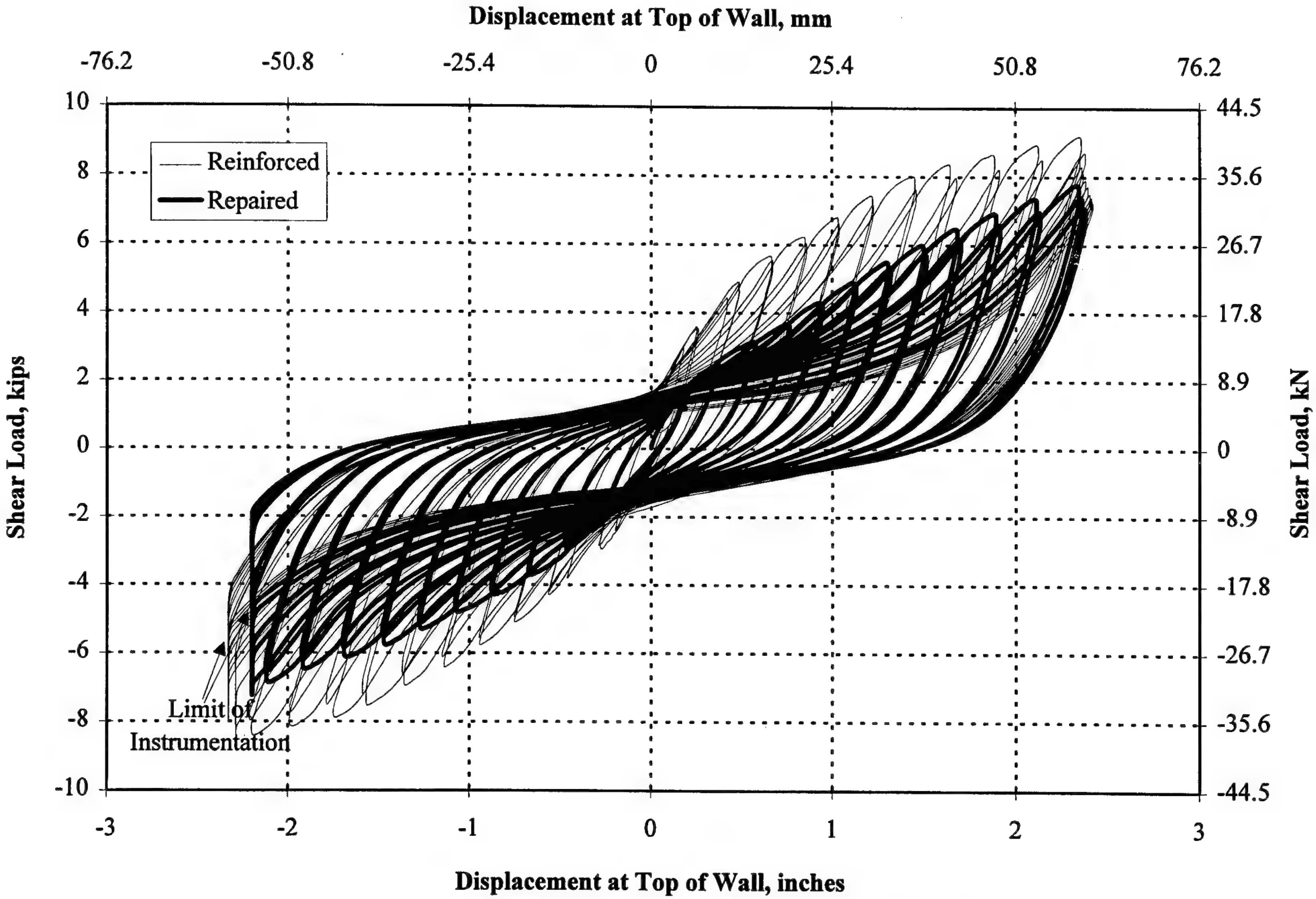

FIG. C10. Total Load-Displacement Response, Repaired Wall

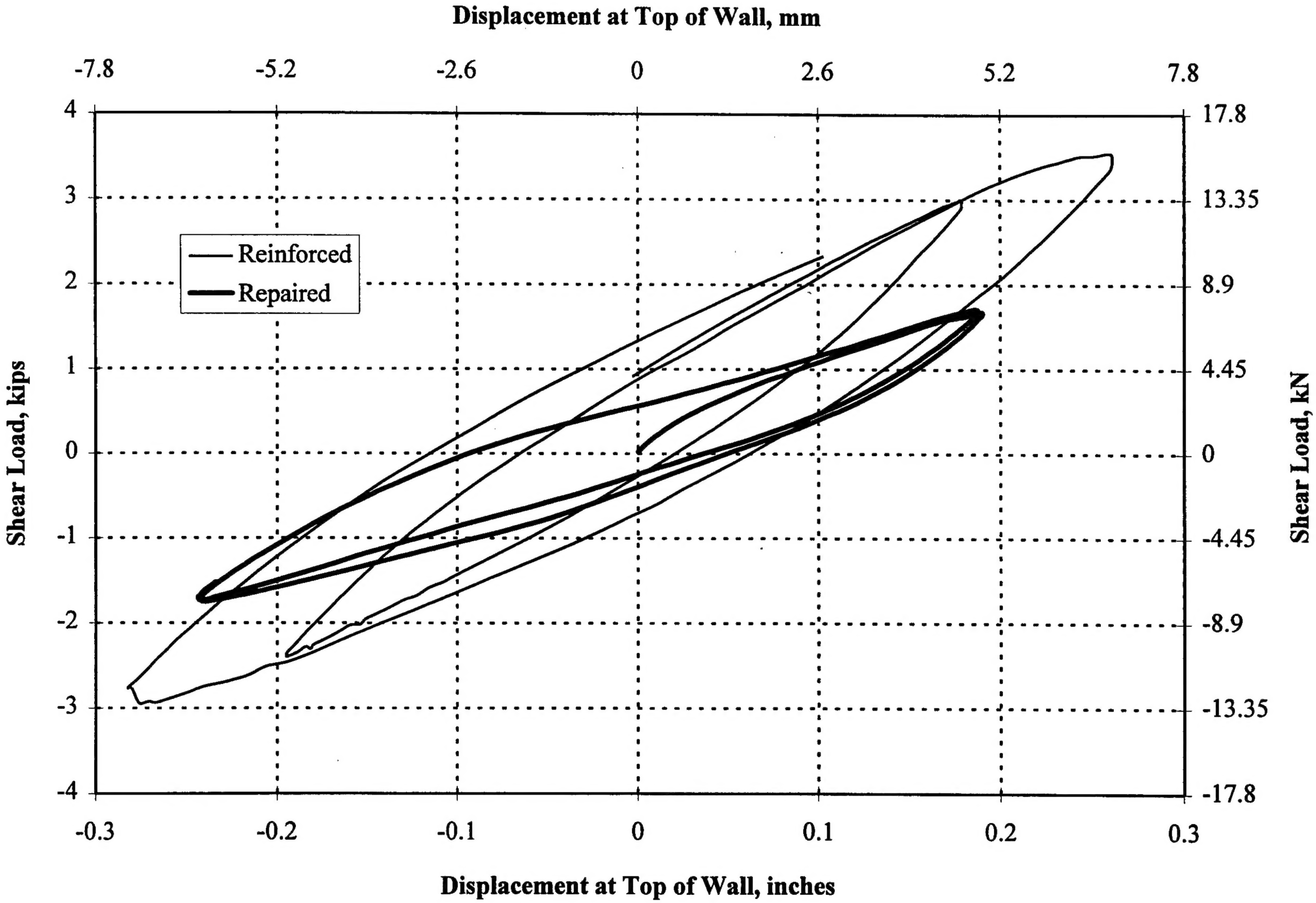

FIG. C11. Repaired Load-Displacement Response, Initial Deflection

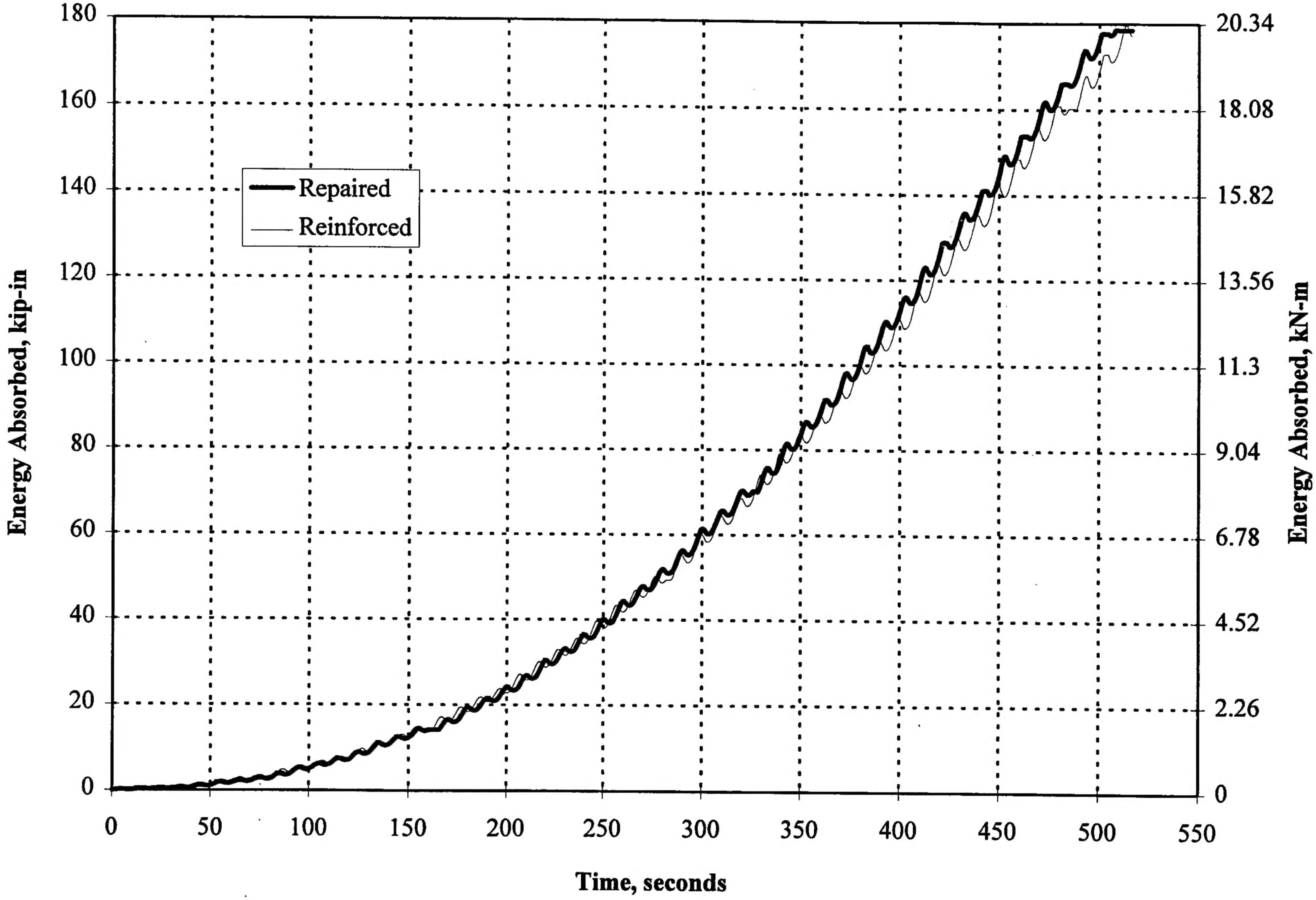

FIG. C12. Energy Dissipated by Wall Before and After Repair

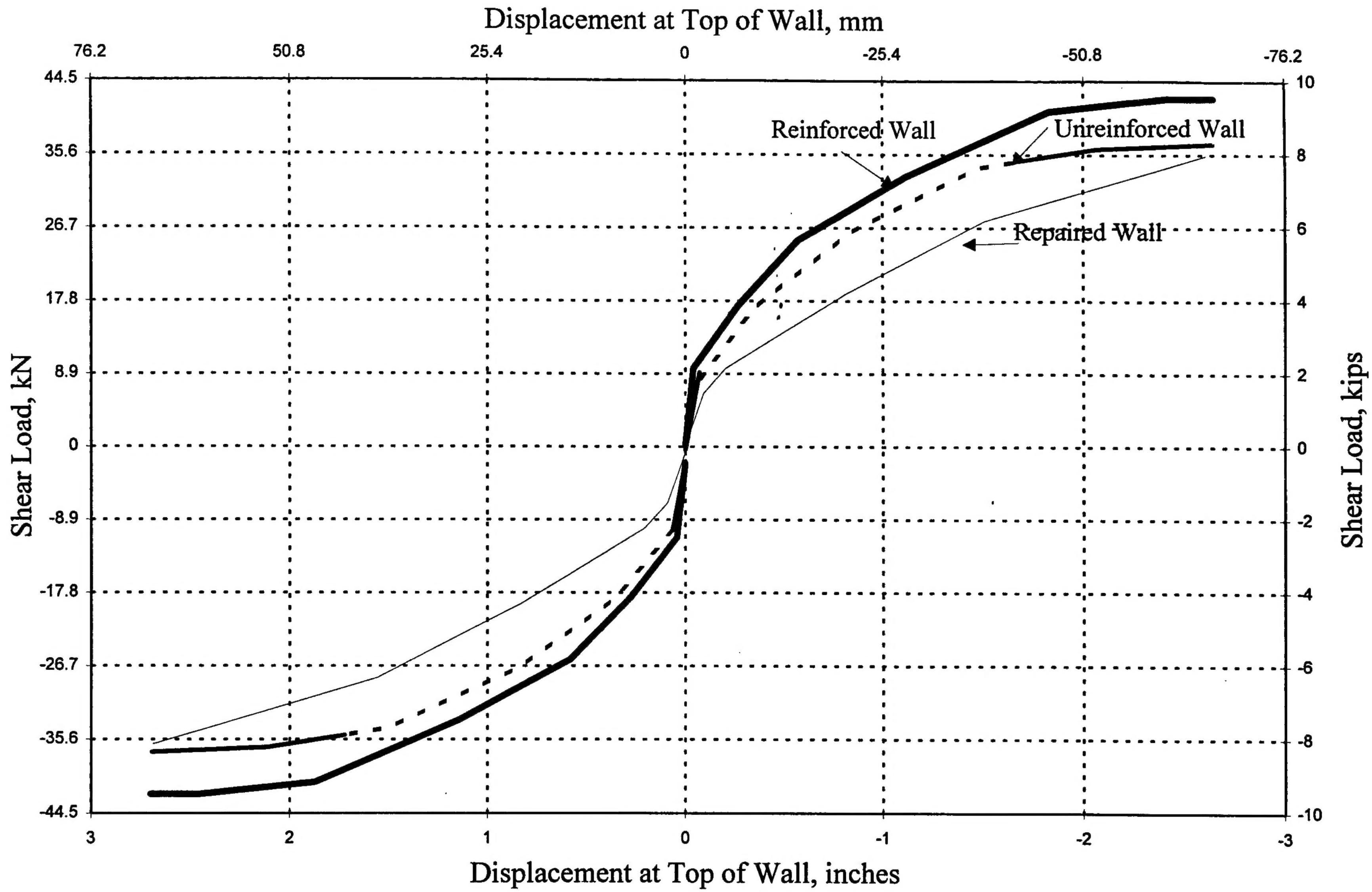

FIG. C13. Load-Displacement Response Envelopes: All Wall Tests

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